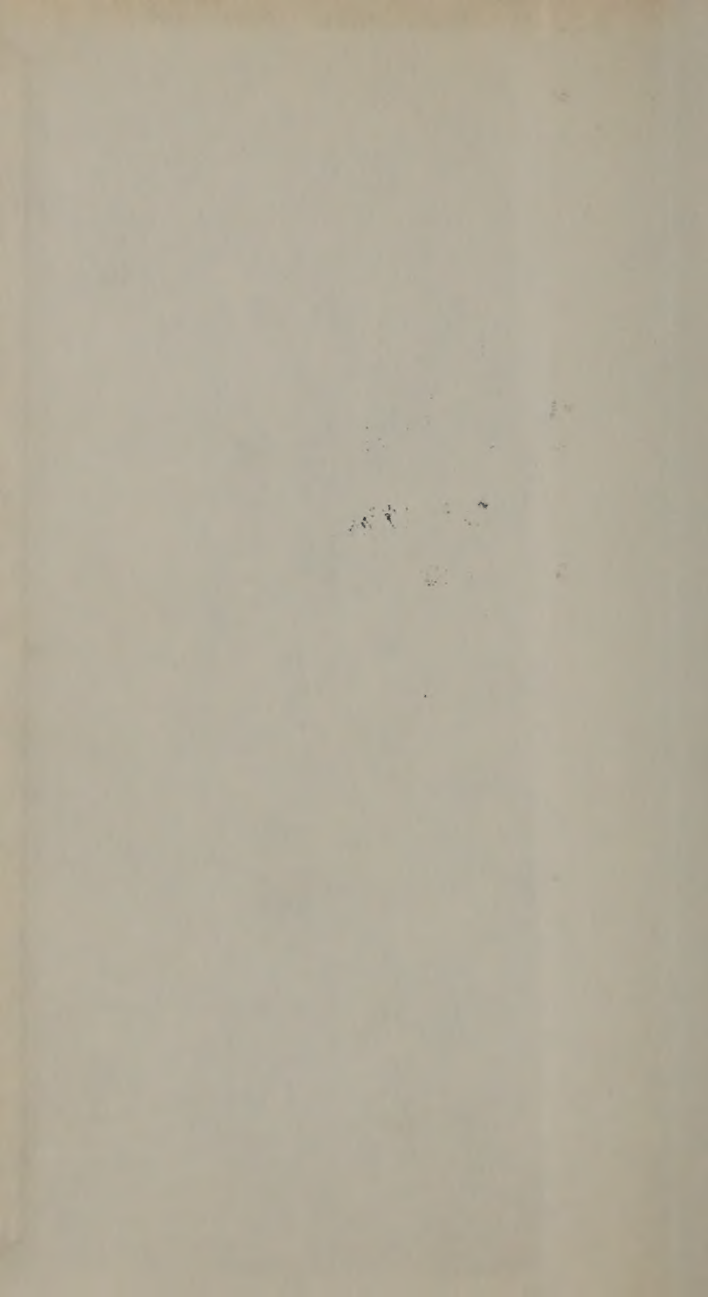


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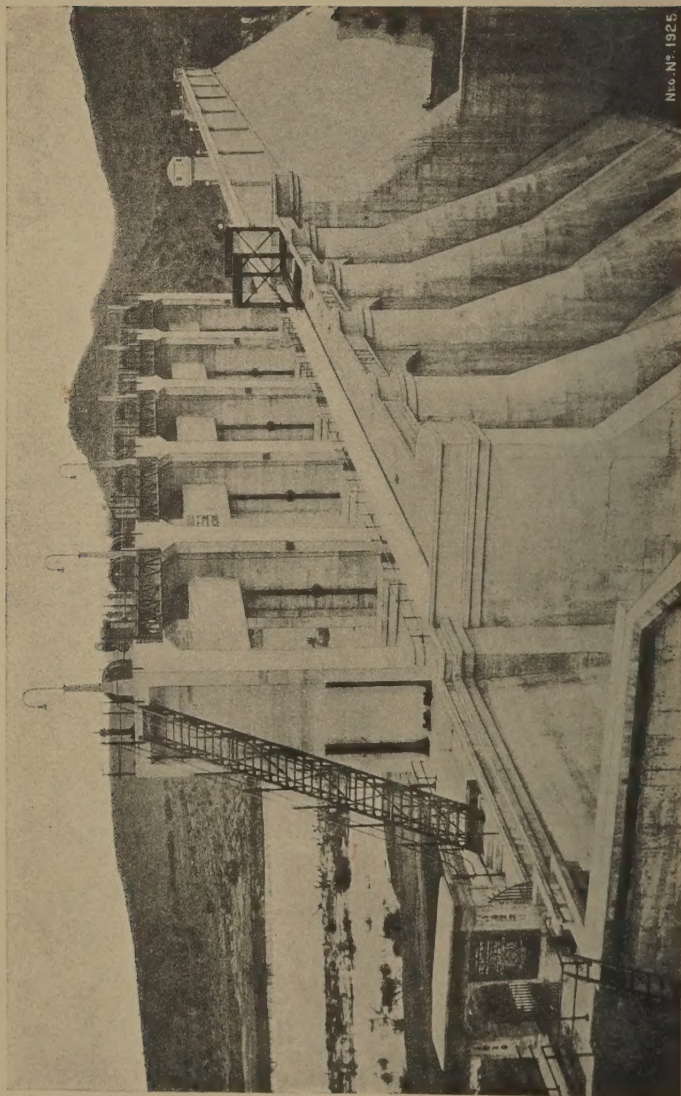
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A MODERN RESERVOIR DAM WITH SLUICE CONTROL.

WATER ENGINEERING

AN INTRODUCTORY TREATISE ON THE
SUBJECT OF WATERWORKS AND THEIR
CONSTRUCTION

BY THE LATE
CHARLES SLAGG

AND BY

F. JOHNSTONE TAYLOR

AUTHOR OF "WATER POWER PRACTICE," "RIVER ENGINEERING," ETC.



LONDON
CROSBY LOCKWOOD AND SON
STATIONERS' HALL COURT, LUDGATE HILL

1929

PRINTED IN GREAT BRITAIN BY
WILLIAM CLOWES AND SONS, LIMITED,
LONDON AND BECCLES.

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PREFACE

IN bringing the late Mr. Slagg's "Water Engineering" into line with modern needs it was decided that the original policy of treating, under one heading, the three distinct subjects of Water Supply, Water Power, and River Engineering, was out of accord with modern practice. There is a good deal of matter in the original treatise that warrants inclusion in the present book, that is, so far as it concerns the matter of Water Supply. Water Power, to-day a subject of supreme importance, is now too much disassociated with Water Supply for treatment in the same small work, and incidentally, the present author treats of the former subject in his "Water Power Practice." That part of original "Water Engineering" dealing with River Works has already been revised and increased to form a separate manual, "River Engineering," while the present work is now brought up to date by the addition of new matter, principally concerning pumping plant and methods adopted to-day for purifying water. These additions, with a judicious selection of the original author's writing, will, it is hoped, render the work acceptable as an elementary treatise on the important subject of Water Supply.

JOHNSTONE TAYLOR

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INTRODUCTION

WHILE this treatise, in the main, is intended to furnish the reader with an outline of the various procedures involved in the carrying out of a water supply of moderate dimensions, the underlying principles are essentially the same whether the supply be for 5,000 people or 500,000. When it is decided that a community has to be supplied with water on a public distribution basis the first procedure is to select the source of supply, of which there is generally speaking the choice of three. That is an upland gathering ground having at its foot a storage reservoir, a river from which the water would be pumped to a service reservoir, or wells. For many reasons the first is to be preferred. The water therefrom is usually of good quality, and if the works are laid out with care, the supply is usually sufficient even during long periods of drought. Circumstances, however, frequently arise when such upland gathering grounds do not exist within reasonable distance, and other supplies nearer at hand have to be considered. In such instances as Manchester, Liverpool, Dublin, etc., where large sums of money would be available, a hundred miles of pipe-line becomes a secondary consideration; but unless a gathering ground is within 20 miles or so of the source of distribution a gravitational scheme may prove impracticable for a smaller supply.

River water usually provides two problems, one involving the pumping of the water to a reservoir from which a gravitational supply can be arranged, and the other means for purifying the water to render it fit for use, a condition of affairs which may also be necessary in the case of upland water. As regards wells, there arises the somewhat difficult problem of deciding just where to sink such wells and in estimating their yield, while if deep the pumping costs may prove a heavy annual charge. Whatever the source of supply most conditions render it imperative to store a certain amount of water at

such elevation that every section of the distribution area shall be supplied with water at a reasonable pressure, say, of 100 ft., and from this reservoir, which is usually distinct from the storage reservoir of an upland gathering ground, the distributing mains and pipes must be laid. It is usual to provide every public street with a main sufficient in size to supply all the premises in it, to maintain a reasonable flow at fire hydrants, and to admit of extension when necessary. The provision of fire hydrants usually devolves upon the water authority, who is also responsible for making the necessary house connections and installing meters, etc., when required.

The water authority is also directly responsible for the purity of the water from a bacteriological point of view, and will have to put down and operate filters, softening plant, or chlorinating apparatus as may be required, as certain standards of purity are required by law.

In the following, the first chapter, Rainfall, the source of all water supply, will be dealt with.

WATER ENGINEERING

CHAPTER I

RAINFALL

THE subject of rainfall and the amount thereof available from any gathering ground is an interesting and absorbing subject which has been written upon by numerous eminent engineers. Of late years American investigators appear to have probed the subject to the bottom, although naturally their conclusions are largely influenced by local climatic and geological conditions. The following notes, in the original form presented by Mr. Slagg, are retained in this revised edition because they are of service to British waterworks men in respect of the tabulated records.

The chief source of the rainfall of this country is the Atlantic Ocean. The heat rays of the sun convert water into vapour; and as the vapour of water is lighter than air, being about five-eighths of the weight of air, bulk for bulk at the same temperature and pressure, it rises into the atmosphere and collects together in clouds, which are blown about by the wind, and chiefly by the prevailing westerly and south-westerly winds, moving laterally and rising up against the hill-sides into a colder region, where it contracts and unites in rain-drops, for as it is heat which changes the particles of water on the surface of the ocean into vapour, so it is the loss of heat which changes the vapour into water again. The vapour itself is invisible; what is seen as clouds is partially condensed into water, but not yet into globules heavy enough to fall through the resisting medium of air.

Rain is precipitated in large quantities on the high ground of the chain of hills which range in a generally north and south direction through England, which shed the water east and west—eastwards into the great basins of the Thames, the

Great Ouse, the Trent, the Humber, the Yorkshire Ouse, the Tees, the Tyne, the Tweed, the Forth and the Tay, and westwards into those of the Clyde, the Eden, the Ribble, the Mersey, the Dee, and the Severn, with other (minor) river-basins; while on the south there are the Avon, the Parret, the Exe, and the Tamar. This dividing ridge begins in the neighbourhood of Devizes, and upon or near it are situated Wootton Bassett, Thames Head, Cheltenham, Daventry, Naseby, Coventry, Birmingham, Wolverhampton, Newcastle-under-Lyme, Buxton, Chapel-en-le-Frith, Saddleworth, Rochdale, Colne, Settle, Ribble Head, Swaledale Head, Tindale Tarn, Tyne Head, Tweed Head, and Clyde Head; and so into the rocky formations of Scotland.

These hills intercept the clouds of vapour and condense a portion of them which is not able to rise over their tops, precipitating a large amount of rain, the fall being generally from 30 in. to 40 in. in a year, and as much as 70 in. or 80 in. in particular situations; while other portions of vapour follow the western slopes until they come to a gap or break in the range of hills, through which they pour and are condensed on the eastern side, giving at these particular places as much rain as on the western side.

If the rainfall on the western coast be compared with that farther inland, it will be seen to increase as these hills are approached. Mr. Bateman drew attention to this in his evidence before the Water Supply Commission, to the effect that if in any particular year the rain on the westerly coast, say, at Liverpool, is 36 in., and going right across the country to the east coast, it increases as the range of hills is approached, where it is at the foot, say, 40 in., on the summit of the hills it is from 50 in. to 60 in.; farther east it is 30 in., and farther east still it is 20 in. In an investigation he had made across England from Liverpool over the Manchester Waterworks drainage-ground and over that of Halifax, it was observed that the winds impinge upon the westerly slope of the mountains which form the eastern side of the Rivington (Liverpool) Waterworks drainage-ground, where in a certain year the rainfall was $48\frac{1}{2}$ in.

Over the ridge, in the first trough to the east, are the Bolton Waterworks, the trough running pretty nearly north and south; and at Belmont, in the Bolton Waterworks district, the rain was 53 in. The next trough is the valley from which the Blackburn Waterworks are supplied, and

there it was 42 in. ; and over the mountains right down in the east it was 30 in.

The Manchester Waterworks* are formed in a mountain range, the Pennine chain of hills, lying between Manchester and Sheffield, commonly called the backbone of England. Rain at Manchester in 1859 was 38 in., in round numbers. Going eastward, the rain at the Denton Reservoir, which is 300 ft. above the sea, was 34 in. ; at Newton it was 35 in., and a little farther east it was 33 in. All these places are upon the plain, or nearly the plain. Then at the foot of the hills the depth of rain was $46\frac{1}{3}$ in. Higher up at the head of the valley, on the west side of the hills, it was $53\frac{1}{3}$ in., then 57·64 in. ; a little farther towards the head, on the east side, just over the summit, it was $58\frac{1}{2}$ in.

At the reservoir of the Sheffield Waterworks,* which is on the hills more to the east, it was 46 in., at Penistone 39 in., and at Sheffield 25 in., showing an increase as the hills are ascended, and, over them, rapidly diminishes towards the east.

On the line of the Rochdale Canal, which also crosses the backbone of England, in the same way, in another year, the rainfall at Rochdale, which is at the foot of the hills, was 39·7 in. At Whiteholme, on the top of the hills, 66·7 in. ; at Blackstone Edge tollbar, 67·5 in. ; and on the east side, 66·6 in. ; but these last three places are almost entirely upon the summits of hills not more than 1,200 ft. high. Then at Sowerby Bridge, which is at the easterly foot, though comparatively in the hills, it was 32·27 in., and at Halifax less than that.

It was observed by Dr. Miller, in the Lake district, that the rainfall increases up to a height of about 2,000 ft. above the sea, but decreases on mountains of higher elevation.

If the elevation of the country lies within the region of the rain-clouds, which may be said to extend to about 3,000 or 4,000 ft., the greatest portion of deposit within that range takes place at from 700 to 2,000, or 2,300 ft. If the greater portion of the gathering-ground lies within that zone, setting aside local circumstances, that will be the elevation which will give the greatest quantity of rain.

Supposing a continuous ridge exceeding 2,000 ft. high to

* The original works. Manchester has now the Thirlmere Supply, and Sheffield is one of the partners with Leicester and Nottingham in the Derwent Valley Works.

range north and south, there would be comparatively little rain on the east side ; it would stop nearly all the rain-clouds from the west, and the water would be precipitated on the westerly slopes ; but, where there is a range of mountains, the summits of which measure upwards of 3,000 ft. with depressions in the ridge, the largest amount of rain falls in the valleys to the east of those depressions. These westerly and south-westerly winds prevail over the greater part of England and Wales ; but in the north of England, in Northumberland, a westerly wind is a dry wind, and a wet wind on the coast of Northumberland is a dry wind in the west.

Mr. Symons began in the year 1858 to collect and arrange the observations which had then been made of the depths of rain fallen in previous years, and to extend the observations, and since 1860 they have been published annually. There are more than two thousand places in this country where the rainfall is observed daily. The observations are examined, collated, and put into form to be useful both in the localities whence they originate and for comparison with the observations of other localities, the whole forming a yearly record of great value.

The rainfall map of the 6th Report of the Rivers Commission, 1874, shows the average annual fall at various places in most of the river-basins. Taking first those on the western coast, which have the heads of the valleys towards the east, the rainfall is seen to increase upwards from the coast as the watersheds are approached, thus :—

					Inches.
Mersey basin :	Liverpool	..	..	..	35
	Manchester	..	..	..	36
	Marple	..	..	..	36
	Chapel-en-le-Frith	..	..	..	40
Ribble basin :	Preston	..	..	..	39
	Settle	..	..	..	50
Lune basin :	Lancaster	..	..	..	44
	Garsdale	..	..	..	52
Lake district :	Whitehaven	..	..	..	52
	Kendal	..	..	..	61

Taking next, from the same records of rainfall, those river-basins in which the heads of the valleys are towards the west, it is seen that in general, but not without exceptions, the depth decreases from the watersheds towards the middle and lower portions of the basins, thus :—

					Inches.
Thames basin :	Cirencester	..	..	..	31
	Oxford	..	..	..	25
	Greenwich	..	..	..	24

				Inches.
Severn basin :	Hengoed	..	..	36
	Shrewsbury	..	..	27
	Worcester	..	..	28
Wye basin :	Rhayader	..	..	46
	Hereford	..	..	30
Trent basin :	Ross	..	..	27
	Birmingham	..	..	31
	Derby	..	..	26
Calder basin :	Retford	..	..	23
	Halifax	..	..	31
Aire basin :	Pontefract	..	..	25
	Head of the valley	..	..	35
	Leeds	..	..	23
Yorks. Ouse basin :	Richmond	..	..	31
	Thirsk	..	..	24
	York	..	..	23
Tyne basin :	Alendale	..	..	50
	Tynemouth	..	..	27

On the " Map of the Rivers and Catchment Basins " of the Ordnance Survey Office a series of rainfall depths are given which, although not the average rainfalls, are comparable one with another, and for this purpose are perhaps better than averages would be. These show the following reductions of depth of rainfall from west to east :—

				Inches.
Thames basin :	Head of the valley	..	..	31·8
	Oxford..	..	..	27·1
	Reading	..	..	24·8
	Windsor	..	..	23·6
	Kingston	..	..	24·2
	Croydon	..	..	24·2
	Greenwich	..	..	28·5
	Sheerness	..	..	22·5
Severn basin :	Head of the valley	..	..	68·4
	Shrewsbury	..	..	23·5
	Worcester	..	..	26·0
	Cheltenham	..	..	25·7
	Gloucester	..	..	21·8
Wye basin :	Rhayader	..	..	41·0
	Hay	..	..	32·7
	Hereford	..	..	28·3
Trent basin :	Birmingham	..	..	31·0
	Leicester	..	..	27·6
	Loughborough	..	..	26·3
Derbys. Wye and Derwent basin :	Head of the valley	..	..	65·1
	Cromford	..	..	37·3
	Belper	..	..	29·9
	Derby	..	..	29·4
Don Basin :	Head of the valley	..	..	54·8
	Barnsley	..	..	27·4
	Sheffield	..	..	32·5
	Doncaster	..	..	33·1
Calder basin ..	..	..	..	49·7, 31·1, and 27·2

						Inches,
Aire basin	..	..	..	..	.. 39·0, 27·7, and	24·2
Wharfe basin	..	..	..	..	.. 54·7, 30·9, „	25·2
Yorks. Ouse basin	..	..	..	..	.. 27·0, 25·0, „	25·5
Wear basin	..	..	..	..	.. 31·3, 25·8, „	24·2
Tyne basin	..	..	..	..	.. 45·5, 28·5, „	24·2

It is only by a patient attention to and collection of observations on the rainfall in various parts of the country that data can be arrived at upon which to base calculations of what quantity of water may be expected to be derived from any particular source in any given locality. All atmospheric phenomena are most intricate in their relations to and effect upon the land, and nothing but the most patient study of observed facts, extending over long periods of time, can determine anything worthy of being relied upon in practice.

Rain-gauges.—The rain-gauge is the instrument by which the quantity of rain falling to the earth is measured. There are various forms of it, from the rude quart bottle with a funnel inserted in the neck, to the more delicate instrument of the scientific professor. Some are made to carry a float in the body of the gauge, to which is attached a graduated rod projecting from the mouth of the funnel, which by its rise indicates the depth of rain fallen. Others have a graduated glass tube attached to the body of the gauge, with a communication between them at the bottom, the water rising to the same level in both.

Others, again, are made with a loose funnel inserted into a vessel which is to be emptied into a separate graduated glass tube when it is desired to know the depth of rain fallen. There are also various modifications of each of these kinds of gauges by the various makers of such instruments. The best size of the mouth of the funnel is not very well determined, but a common size is 8 in. diameter; others, again, are 10 in. diameter, but Mr. Symons, who has under his control a great number of rain-gauges in the kingdom, and who has paid a great deal of attention to the niceties of measurement, said that a funnel of 5 in. diameter is as good as any other, and sufficiently accurate.

The height at which a rain-gauge is fixed above the ground is important to be attended to. It has long been known, but apparently not generally known, that the nearer the ground the gauge is set, the more rain it registers, but it was not until Mr. Symons's comparative experiments had been made,

that anything like a definite conclusion on this point had been arrived at. Gauges were fixed at all heights from the surface of the ground to a height of 20 ft.

At the ground-level itself the cause of the greatest register of quantity was proved to be because of the rebounding into it of rain-drops falling on the surrounding soil or grass. When, however, a trench was dug round the gauge so as to prevent this, the register ceased to show excess. On the whole, Mr. Symons recommends the mouth of the rain-gauge to be not less than 6 in. or more than 12 in. above the ground, except when a greater elevation is necessary to obtain a proper exposure; for this is important.

Gauges sheltered by buildings, or trees, or shrubs, or tall flowers, are not in the best situations. The distance from any building should be at the very least as great as its height. The gauge must be fixed perfectly level. If it be impossible to fix the gauge unless near some buildings, those are to be preferred which stand north-west, north, and north-east of the gauge; those standing south and south-east will be in the next least objectionable quarter, and those standing south-west of the gauge are in the most objectionable positions. Observations should be made daily, and at the same hour of the day.

In snow, that which is caught in the funnel should be taken out and melted and measured as rain. As a check upon this, select a place where the snow is of a fair depth, not drifted, invert the funnel and take up a funnel full, or whatever can be taken up in this way, and melt it. As a second check, measure the average depth of snow and take a twelfth of it as the equivalent depth of water. Strike an average of the three processes and enter that as the depth of rain. But it is to be observed that in respect of the first of these methods the snow is liable to be blown out of the funnel if the weather be at all windy, and allowance, therefore, should be made for this. Snow, measured in depth within a few hours after it has ceased to fall, has often been found to yield, by melting, a depth of water equal to one-twelfth of its depth, not infrequently one-tenth, and sometimes one-eighth; it depends upon the state of the atmosphere at the time of the fall.

CHAPTER II

EMBANKMENTS OF STORAGE RESERVOIRS

RESERVOIR embankments in general are made with the materials found on or near the site, be they earth, shale, or stone. With earth or shale the English practice makes the watertightness of a reservoir dependent on a wall of puddled clay, carried up in the centre of the embankment from retentive ground below its seat. The thickness of the puddle wall varies from 4 ft. to 8 ft. at the top, and at the bottom it is as much more as is required for a batter on each side of an inch to every foot in height. But it is not good practice to make the thickness of the puddle wall at the top so little as 4 ft., and 6 ft. is the least dimension that ought to be given to it, increasing beyond this, according to the quality of the clay, to 8 ft. in perhaps most cases. Thus, where the height of the bank is 24 ft., the thickness of puddle at the bottom would be 12 ft., or half the height. Where the height is 48 ft., the thickness would be 16 ft., or one-third of the height; and it has been recommended to make the thickness one-third of the height also in high banks, and this would be, for a bank 96 ft. in height, 32 ft., although, according to the proportions above stated, it would be 24 ft., or, in this case, one-fourth of the height.

Puddle.—Puddled clay, upon which the watertightness of reservoir embankments has for so long a time depended, since, at least, the time of Brindley and canal-making, has come to be regarded with disfavour and distrust where it is itself exposed to the action of water or of the air. When some canal-makers complained to Brindley that they could not stop a run of water, his advice was to “puddle it, puddle it”; and the good reputation which puddle has always had in waterworks-engineering led at one time to its careless use, and it was solely depended upon for the watertightness of reservoir embankments, which, however, was not always effected.

Certainly, some reservoir embankments have been made of very great height, and they have stood safely, and there is no apprehension whatever that they will not always continue so, with, of course, due attention to the outer parts, to repair the continued action of the weather—frost, thaw, rain. But these embankments have been well made, not only in the core, which is formed of puddled clay from end to end of the bank, like a wall, but on each side of it; on the inside to prevent the water in the reservoir having access to it in any considerable body, and on the outside to prevent the water with which the puddle is made being drawn out of it, whether by evaporation or capillary attraction, by which it might become gradually dried, and the cracks which would be consequently formed might extend so far into the wall as to reduce its virtual thickness materially. Rough stone, for instance, or earth loosely tipped in, would admit the air to it, and the water incorporated with the puddle would continue to evaporate as long as the air itself was not saturated with moisture.

Puddled clay does not readily part with the water with which it is incorporated, and not at all if the outer and drier air is wholly excluded from it; but this can only be done by placing against it a compact material. On the other hand, in the presence of water having any motion, it easily dissolves—melts away; and the best puddle for a wall—that is, the closest in texture—melts the soonest under this action; nevertheless, it is absolutely necessary that puddle should be so worked as to bring it to a close texture; but at its best it is porous, and if water has access to it under great pressure, it is only a question of degree how much water will be forced through the puddle wall. In the first place, a great deal of water is used in making it; dry clay will absorb nearly a third of its own weight of water; and if it be taken, not in a purposely dried state, but in a naturally dry state, it will absorb, in the process of puddling, say an eighth or a sixth of its weight of water. Thus, water acting under pressure on one face of a puddle wall tends to force water out on the other side, being itself incompressible, and the quantity it can force through in a given time is limited only by the smallness of the pores and the length and tortuousness of the course which a run of water must follow. It is, therefore, highly important to protect the puddle from contact with water under great pressure. It might almost seem that if it requires

so much protection it can be of little use, and that it might be dispensed with altogether ; but this extreme would be as unwise as that of depending solely upon it without precautions ; and as it is so readily procurable, it remains still a necessary material for waterworks reservoirs.

There are two kinds of puddle—clay puddle and gravel puddle—proper to be used for different purposes, or rather in different positions for the same purpose of watertightness ; the one consisting of clay only, the other with stones incorporated. When there is a liability to a wash of water, stones are necessary to hold the clay together, and of stones rounded gravel-stones are the best, besides being generally the more easily obtained, inasmuch as they are more uniformly dispersed through the mass of clay than angular stones can be in the process of working, which is by cutting and cross-cutting with long-bladed tools, reaching down at every stroke, through the layer of clay being worked, into the puddle beneath it. The tool slips past a rounded gravel-stone without much disturbance of the adjoining clay ; but angular stones, when struck by the puddling tool, have a tendency to congregate together. Gravel puddle is proper to be used where weights have to be sustained, as for the walls of a service reservoir. Clay puddle can hardly be worked stiff enough to support the weight, which squeezes the clay outwards and upwards, and there is an inconvenient settlement of the wall ; but with gravel puddle the weight can be sustained and watertightness effected also, because with a mixture of gravel the puddle can be worked stiffer.

Form of Bank.—Figs. 1, 2, and 3, show respectively a longitudinal section, a plan, and a cross-section of an embankment formed of earth, with a puddle-trench and wall. The trench is shown to be cut straight down, so that the bottom is as wide as the top, and the advantage of this over sloping sides is that, in case the ground should not prove watertight at the depth anticipated, the trench may be carried down to any further depth. The trench is filled in with puddled clay up to the surface of the ground, above which, up to and above the top-water level of the reservoir, the puddle is continued as a wall, having a batter on each side of 1 in. to a foot, and finishing 8 ft. thick at the top ; 4 ft. is not a sufficient thickness, 6 ft. is sufficient with precautions, and 8 ft. is advisable ; if, however, there be the means of placing a nearly watertight mass of earth next

the puddle, as BB on the sketch, the thickness of the top of the puddle may be 6 ft.

In the longitudinal section, Fig. 1, the bottom of the puddle-trench is cut into the hill-side as far down as to pass



Fig. 1.

through loose or fissured ground to a watertight bottom, which is often found nearer the surface on one side of the valley than on the other. That side should be chosen for the discharge-pipe or culvert, but it cannot always be determined before the puddle-trench has been cut.

On the plan, Fig. 2, AB is the original course of the stream ; CD, the two ends of the embankment ; EF, an open channel,

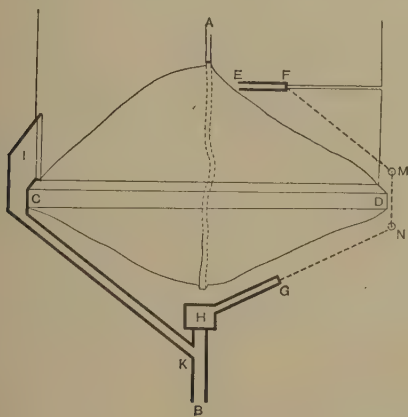


Fig. 2.

cut near the bottom of the reservoir ; FG, the discharge-pipe, laid in the solid ground ; sometimes laid in a culvert of masonry, itself laid in the solid ground. GH, an open channel ; H, the gauge-basin ; I, the waste-weir ; IK, the bye-wash.

The discharge-pipe has a valve on its inner end, at F, and it is necessary to erect a tower of some kind, from the top of which it can be worked. Access to the top of the tower is

procured by means of a bridge, either from the top of the embankment or from the side of the reservoir, as shown in the sketch. When the puddle-trench is everywhere sunk below the level at which the water is drawn off, the discharge-pipe is taken round the end of the bank, instead of direct from F to G, for the sake of being in the solid ground, and a tunnel is then preferable to an open trench, as shown by the dotted line F, M, N, G, shafts being sunk at M and N. It may not be quite justifiable to put the preference of a tunnel on the less disturbance of the ground, for with careful timbering perhaps less disturbance would occur with an open trench. The open trench can, at least, be closely filled in again, which cannot be said with certainty of the space between the crown of a tunnel and the earth above it.

Outlet.—Although tunnels have been advocated instead of culverts, it would certainly be unwise to adopt them in every case. In every case, however, there is a necessity for a tower or valve-well, for a valve cannot be satisfactorily worked with sloping rods, and because it is absolutely necessary to place a valve on the inner end of a discharge-pipe, if any regard at all be had to the safety of the reservoir. Certainly, discharge-pipes have been laid under embankments, which have had no valves at the inner ends, the discharge being controlled at the outer ends, at the foot of the embankment; but considering that a cast-iron pipe, if of large size, may be broken by the external pressure of the earth, and that, whatever the size, it is subject to leakage at the joints, where it cannot be got at for repair, this method will not bear repetition. A valve, then, being necessary at the inner end of the discharge-pipe, and a tower for access to it, it becomes desirable to draw off the water, not from the bottom of the reservoir, where it is muddy, but as near the top at all times as is practicable, and the tower affords facilities for the insertion of a short pipe through the wall at as many different heights as may be desired.

Instead of placing the tower or valve-well in the reservoir at or near the foot of the slope of the embankment, it has been sometimes placed almost in the middle of the bank, close to the puddle wall, and the water is conducted to it by a culvert from the foot of the inner slope of the embankment; but this position does not afford the facilities of drawing off the water at different heights that the exposed tower does.

With a good masonry culvert or tunnel of sound bricks or

thick stonework, a pipe in addition seems superfluous, where the reservoir is constructed for one discharge only ; but if it be so situated that two discharges are required from it, one, for instance, for a town supply, and another for the stream itself, the continuation of the supply-pipe up to the valve-tower, through the culvert or tunnel, is necessary, the discharge to the stream running on the bottom of the culvert or tunnel beneath the supply-pipe, which is supported from side to side so as to leave room beneath it for the stream discharge. The quantity given to the stream, under these circumstances, is usually required to be open to inspection by turning it over a gauge-weir to a depth agreed upon, the length of the gauge being fixed, by which the quantity being discharged can be ascertained for the satisfaction of millers and others interested in the stream.

The waste-weir and bye-wash are shown in the sketch on the opposite side to the other works, but, according to circumstances, they may be on either side. They are very important parts of the work, for when the reservoir is full at a time of flood nearly all the water must pass that way, and therefore not only must the length of the waste-weir be sufficient to prevent the flood rising to a dangerous height against the embankment, but the bye-wash must be so constructed that it will not be torn up by a great body of water rushing down so steep a channel.

In the cross-section, Fig. 3, A is the puddle-trench and wall ; BB, material selected from that excavated as being

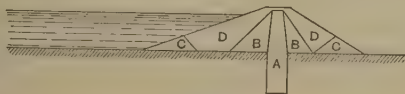


Fig. 3.

the most easily compacted ; CC, a rough stone bank necessary for the outer slope, and advisable also for the inner ; DD, the rest of the material excavated, except peat. This is a material which, although sometimes abundant on reservoir sites, is very objectionable in an embankment, even in small quantities.

The top of the embankment, when consolidated, has usually been made from 4 ft. to 6 ft. above the top-water level, and in situations much exposed to high winds not less than 6 ft. ; for a gale of wind blowing down the reservoir rolls

against the face of the bank waves of considerable height, and this may occur when the reservoir is more than full, and water going over the waste-weir at a depth of, perhaps, 2 ft., thus reducing the clear height to 4 ft.

Safety.—The width of the top of the bank has usually been made from 12 ft. to 24 ft. The proper width is determined by the considerations of whether or not a roadway is required to be made along it, and what width the puddle wall is to be at the top. Leaving out of consideration the question of roadway, which must be always a local requirement, the width necessary to properly enclose and protect the puddle wall is about twice the width of the top of the puddle wall itself.

The safety of a reservoir embankment does not depend upon one thing only, as upon the perfection of the puddling, but equally upon several others, chief amongst which is the consolidation of that part of the bank which lies within the puddle wall, and into which, therefore, water would penetrate if not prevented, and this prevention depends chiefly upon the thickness of the layers of earth with which the embankment is raised. Thin layers are always desirable; but the maximum thickness which may be allowed depends on the material and the means by which it is deposited in the bank. A heavy and dry material may be deposited in thicker layers than one of less specific weight, with equal effect in both cases in forming a solid bank. The thickness allowable has been variously stated at 6 in., 12 in., 18 in., and 2 ft.

Where the chief consideration has been the urgent progress of the work to meet a demand for water, as was the case when, in order to meet the demands of the approaching summer the embankment of the reservoir on the Dale Dike at Bradfield was urged forward by the Sheffield Waterworks Company, in the early part of the year 1864, and which burst in March of that year; the earth forming the bank on each side of the puddle wall had been tipped from rail-waggons at a height of 5 ft. or 6 ft., and more; but few waterworks engineers would sanction that, except for the outer part of the bank, and even for that part it is too high, because the large and small rock are not evenly mixed; the large lumps roll to the bottom of the tip and form a layer there of themselves; the finer stuff lying together upon the coarse layers; whereas, to prevent slipping, which is one great object of all the precautions, the large and the small should be as evenly mixed as is practicable, except in those parts where material

is purposely selected to be deposited, fine in one part and large in another, for different purposes. If the material tipped into the bank were all stone, the rolling of the large lumps into a separate layer would be less objectionable; and, indeed, with stone for the material it might even be an advantage that the layers should be so arranged, for the one would drain the other, and the bank, as a whole, would retain less water than it otherwise would do; but with other materials, such as shale, whether blue or dark, or, indeed, any material but stone, the same object would not be effected.

In making the inner part of the embankment of a reservoir the object is to make it as compact as possible to prevent the water reaching the puddle wall, and, therefore, the material should be small and clayey; but in the outer part of the bank dryness and stability are the chief objects, for which larger

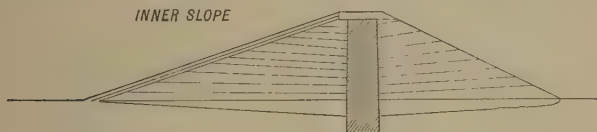


Fig. 4.

and harder material is more suitable. The inner slope of an embankment is usually made flatter than the outer slope, as 3 to 1 inside and 2 to 1 outside, for the inner part of the bank is more liable to slip than the outer. When the water-level in the reservoir is reduced, if the inner part of the bank has been penetrated by water and become partially saturated, slips are more likely to take place, and the flatter slope meets this tendency; but outside, with dry materials, 2 to 1 is as good a slope as 3 to 1 inside. As a further precaution against slipping it is desirable to keep up the outer parts, next the slopes, higher than the inner parts next the puddle wall, as in Fig. 4.

CHAPTER III

MASONRY DAMS

MODERN practice is largely in favour of masonry and concrete dams for water-supply work.

There are in the main five distinct types, viz. :

- (1) Small mass dams.
- (2) Large mass dams.
- (3) Arched dams.
- (4) Buttressed dams.
- (5) Multiple arch dams.

In the case of small dams a trapezoidal section is usually the most suitable. The curved profile given to high dams is the result of rather elaborate mathematical investigation into the stresses involved, and these show that a reduction of the section from the trapezoidal may be made without affecting the stability. In a large dam this may enable a large reduction of material to be effected, but this saving does not become apparent in a low dam.

The general principles underlying the design of trapezoidal

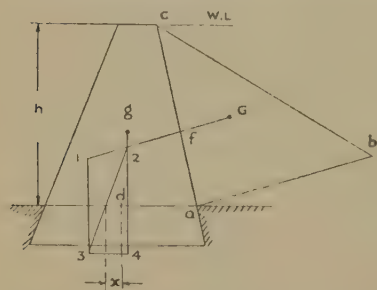


Fig. 5.

dams are illustrated in Fig. 5. The side *ac* is exposed to water pressure. Considering a layer of the water 1 ft. in length *ab* is drawn at right angles to *c* and of a length equal to *h*. Completing the triangle by the line *bc* there is a triangular prism of water weighing $62\frac{1}{2}$ lb. per foot cube exerting pressure against the wall. If *G* is taken as

the centre of gravity of this prism, the line *gf* perpendicular to *ac* indicates the pressure of the water and its direction. *g* denoting the centre of gravity of the wall, whose weight obviously acts

downwards, the parallelogram 1, 2, 3, 4 can be drawn, making 1, 2 equal to the water pressure, and 2, 4 equal to the weight of the wall. When completed—the diagram must be accurately drawn to a large scale—the diagonal 2, 3 must, in order to ensure safety, fall within the middle third of the walls. A few trial sections will enable a suitable one to be arrived at. The diagram, however, merely shows that the section chosen is, by the principles of mechanics, safe from overturning because the resisting moment of the wall is greater than the overturning moment of the water. This, of course, is the essential principle of the design of every mass dam. In addition, it is necessary to investigate what tension there is at the inner side of the base and what compression exists at the outer side of the base when the wall is under pressure. In Fig. 5 the point d is assumed to be the centre of the base and the distance x must be scaled off. Let W be the weight of the wall per foot run, and A the area of the base 1 ft. wide.

Then calculating M equal to Wx and Z equal to $\frac{1}{6}BD^2$, B being the width and D the thickness of the wall— B is usually taken as 1 ft.—there is an expression for calculating tension and pressure at the base reading

$$\frac{W}{A} \pm \frac{M}{Z}$$

Working this expression out with the plus sign gives the compression at the toe, which must be within safe limits, while using the minus sign shows if any tension exists. If it does it may be necessary to introduce reinforcement along the inner side of the wall or to re-design the structure.

This simple expression is very useful in dam design, and may be explained as follows :—

I is the moment of inertia of the section ;

y is the distance of extreme fibre of cross-section from the neutral axis, which passes through the centre of gravity of the section ;

p is the unit stress at the extreme fibre produced by the bending moment ;

then, according to the principles of mechanics,

$$p = \frac{M}{I} y$$

If p_1 = unit stress produced by W ,

$$p_1 = \frac{W}{A}$$

where

A = cross-section area.

Therefore total stress at the given point in section

$$= \frac{W}{A} \pm \frac{M}{I} y$$

The plus sign applies when M produces compression at the given point, the minus sign when tension. Many text-books denote $\frac{I}{y}$ by the symbol Z , and term Z "the section modulus," since for any chosen section it is a constant.

It must be remembered that there will be two values for Z , say Z_c and Z_t , the first for the side of the neutral axis in which compression is produced by bending and the other for the tension side, and so

$$Z_c = \frac{I}{y_c} \text{ and } Z_t = \frac{I}{y_t}$$

where y_c and y_t are the distances of the extreme fibres on the compression and tensile sides of the section. In symmetrical figures, such as foundations, this correction will, of course, not be necessary.

The addition and subtraction of the two sides of the equation gives the compression and tension respectively.

Large Mass Dams.—While based upon elementary principles the design of large dams becomes rather an involved process when the two conditions of stability and minimum cross-section have to be satisfied. It is only possible here to outline the methods involved in a very brief manner.

Numerous formulæ are quoted for obtaining a suitable section for dam structures of any practical height, but those propounded by Rankine are given here. Rankine bases his design on the assumption that at any depth d' below the surface equal to 120 ft. the thickness T is equal to 84 ft. The curves of the faces are logarithmic, and a constant sub-tangent to the two is given by Rankine as 80 ft. equal a .

With these values assumed, at any depth d the thickness t is found by the expression

$$\log t = \log T + .4343 \frac{d - d^1}{a}$$

Assuming a top thickness of 18.74 ft. the sectional area of any section of the wall measured from the top of the wall

$$A = \int_0^d t D d \\ = a(t - t)_0$$

To draw the lines of resistance for the reservoir full and empty the following values are required :—

The moment of horizontal pressure of the water at any point on the weir at depth $d =$

$$M = \frac{d^3}{12}$$

Knowing A as found above,

$$\frac{d^3}{12} \div A$$

gives the horizontal distance x between the lines of resistance for reservoir full and reservoir empty, that is to say,

$$\frac{M}{\int t d^3} = \frac{Dd}{12a(t-t_o)}$$

on any plane D feet below the top

$$p = \int \frac{t D d}{t} = a \left(1 - \frac{t_o}{t} \right)$$

or if R = the deviation of the line of resistance from the middle of any horizontal plane, the maximum value of

$$p = a \left(1 - \frac{t_o}{t} \right) \left(1 + \frac{6R}{t} \right)$$

where R is the above deviation.

As the above form of dam is not suitable for small depths it is usual to consider a high dam in two sections : that above and below 110 ft. For the portion below 110 ft. the above reasoning applies ; for that above 110 ft. reference may be made to Fig. 6. The width of the base is divided into two portions, A and B. Assuming a height H in feet and with P the limiting compressive stress on the material

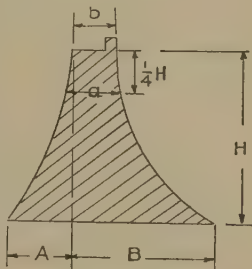


Fig. 6.

$$B = \sqrt{\frac{0.05H^3}{P + 0.03H}}$$

$$A = \left(\frac{0.09H}{P} \right)^4$$

The minimum value of B should not be less than 0.6H. The curve of the face of the wall is obtained by working out several

values of B for selected values of H . When near the top a distance $= \frac{H}{4}$ may be set off, and the value of a is found by the expression for finding B , only using $\frac{1}{4}$ of the value of H .

In the application of formulæ, however, it may be said that there are so many factors which enter into the final choice of section for a high dam that several trial sections would probably be got out by alternative methods, and compared with existing structures of a similar nature. Here, in fact, a study of completed works is the best guide.

Arch Dams.—A deep, rocky valley provides a situation where a dam constructed as an arch between the two sides may probably cost less than a mass dam. There are not a great many of these dams in existence. Several dams which are curved in plan are mass dams to all intents and purposes. Erected in 1884, the Bear Valley Dam in California is still regarded as a bold venture, having but a thickness of 8 ft. at a depth of 48 ft. below the crest. There is no doubt about its being an economical form of construction when conditions are suitable, but many engineers still regard it as a hazardous venture, and proposals on foot in America to test to destruction a full-sized structure of this class will no doubt serve to enlighten engineers just how much the section may be reduced by arching. It is only under certain circumstances that any real economy can be effected by this construction, and this depends upon a relatively small radius being feasible. Bearing

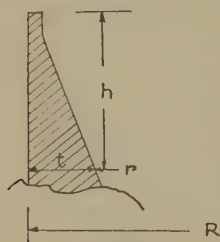


Fig. 7.

in mind also that a valley is naturally vee shaped, the radius may be reduced at the lower parts of the dam. A trial section may be got out by the following formulæ taken in conjunction with Fig. 7.

If R is a selected radius for the water face, and r the radius at any other point at the same level, then w being equal to the weight of 1 cubic ft. of water, and R and h as shown, then: The horizontal thrust at every point of that particular curve is equal to Rhw . Denoting the limiting stress on the masonry by s , then as $R-r$ equals t ,

$$t = R \left(1 - \sqrt{1 - \frac{2hw}{s}} \right)$$

or assuming R equals 100 and h equals 100, $w=62.5$ and $s=10$, then

$$t = 100 \left(1 - \sqrt{1 - \frac{2 \times 100 \times 62.5}{10 \times 2240}} \right) \\ = 34.3 \text{ ft.}$$

Buttressed Dams.—The extended use of ferro-concrete has given considerable impetus to the use of buttressed dams. For structures of moderate height it is likely to show considerable economy in material. Referring to Fig. 8, the first procedure is to fix upon the batter of the outer face, and the value of x is required. Using the notation as shown on the diagram, and assuming the ratio of clear space between the buttresses to their thickness to be denoted by r , the value of x is

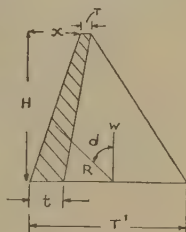


Fig. 8.

$$= \frac{T_1^3 + rT^3}{T_1^2 - rt^2} - S \left(\frac{T_1 + rt}{2r - 1} + T \right)$$

S being the specific gravity of the material used. In order to find x , however, which is the minimum safe value, values for t , T , and r have to be assumed as follows :

$$T = 0.5 + 0.1H$$

$$t = T + 0.05H$$

$$r = \frac{50 - H}{10}$$

B , the thickness of the buttresses, may be taken as $\frac{1}{6}H$. T_1 can only be solved by a very complex expression, and some tabulated values are given hereunder.

With the requisite dimensions got out, it is necessary to prove whether the maximum pressure at the toe is within safe limits, and that the resultant does not make an angle with the vertical greater than the angle of repose. The value of α may be found from the expression

$$\cot \alpha = \frac{(r-1)(x+ST) + S(T_1+rt)}{H(r+1)}$$

and

$$P = 0.028H^2(r+1) \frac{T_1 \cot \alpha \sec^2 \alpha}{T_1^2 + rt^2}$$

Having found a value for α , the previously obtained value for x may be checked by the expression

$$x = H \cot \alpha - ST - \frac{S(T_1 + rt)}{r + 1}$$

The maximum value for α is about 40° .

The tension in the dam between the buttresses can be easily computed by assuming it as a loaded and supported beam between the buttresses. This construction in plain concrete will effect a saving of some 35 per cent. of material as compared with a mass dam of similar height; in ferro-concrete the saving would be considerably more. In fact, this class of dam is very suitable for being designed as a ferro-concrete dam, and especially is this the case if the slabs between the buttresses are considered as arches.

TABLE OF VALUES REFERRED TO IN ART. 15.

5	..	..	..	5.90	16	..	..	..	18.25
6	..	..	..	6.80	17	..	..	..	18.75
7	..	..	..	7.90	18	..	..	..	20.00
8	..	..	..	9.0	19	..	..	..	22.00
9	..	..	..	10.0	20	..	..	..	23.25
10	..	..	..	11.40	21	..	..	..	24.50
11	..	..	..	12.50	22	..	..	..	25.75
12	..	..	..	13.55	23	..	..	..	27.00
13	..	..	..	17.70	24	..	..	..	28.50
14	..	..	..	15.90	25	..	..	..	30.00
15	..	..	..	17.00					

Multiple Arch Dams.—The flat slabs of the class of dam just outlined naturally suggest the use of arches between the buttresses, the latter acting as their abutments; and this form of construction has, in fact, of late become very general. It is essentially suited to ferro-concrete, and as this branch of engineering is a specialised business, it will suffice to describe here briefly a completed dam of this class. The buttresses, usually of a thin, tapering section, are spaced about 40 ft. apart on an average, and have the openings between them closed by a series of arches facing upstream. The buttresses, which are on the downstream side, are connected by ferro-concrete ties. The resulting structure will be seen at once to be very economical of material by permitting of the use of very slender sections, yet at the same time, it is one which, examined in the light of a structure, has considerable resistance along the lines which hydrostatic pressure calls for such resistance.

An interesting example is seen in the Great Lakes Dam,

Tasmania, in connection with the comprehensive water-power developments in that island. Impounding water to a depth of some 40 ft., the dam has a total length of some 1,080 ft. with a maximum height of 45 ft. It is formed with 27 arches at 40-ft. centres. A range of outlets is built into the centre of the dam in place of the more usual spillway, and the general construction will be best understood by considering it in three portions, that is, Arches, Buttresses, and Outlet.

The Arches.—Fig. 9 gives a general section of one of the arches, from which it will be noticed that the face has a slope

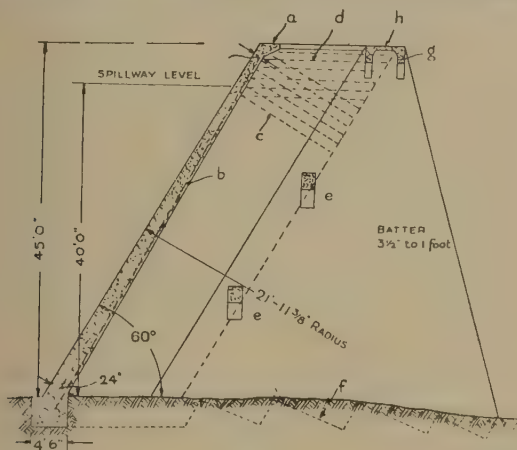


Fig. 9.

of 60° . The thickness of the crown at the top is 12 in., although it will be observed that there is a lip added at *a*. The thickness is gradually increased to 24 in. at the foot, while a toe of the usual form brings the footing *j* to a width of 4 ft. 6 in. The external radius of the arches is 21 ft. 11 $\frac{3}{8}$ in., the internal radii being 20 ft. 5 $\frac{5}{8}$ in. at the base and 20 ft. 6 $\frac{5}{8}$ in. at the top. The reinforcement is disposed on the underside of the arches, as shown at *b*, extra rods being introduced at the haunches. The latter are arranged 3 in. below the upstream face normal of the 60° inclination. Round rods were used for the whole of the reinforcement, in accordance with modern practice, which tends to make use of round bars in preference to any special sections, those used in the haunches of the arches being $\frac{3}{4}$ in. dia. and variously spaced 7 in. to

12 in. according to the depth from the top, while $\frac{1}{2}$ -in. bars are used for roughly the middle third of the arch. As regards longitudinal or circular reinforcement, this mainly consists of

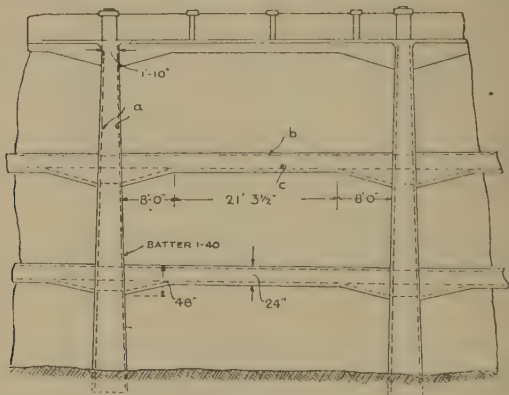


Fig. 10.

$\frac{1}{2}$ -in. bars at 12-in. centres normal to the upstream face, as at *c*, except for 6 rods, *d*, at the top, which are horizontal.

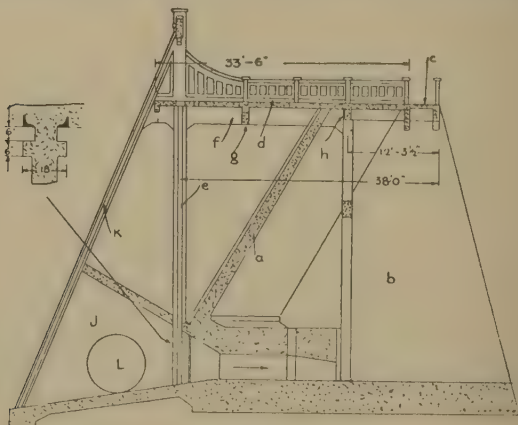


Fig. 11.

Buttresses and Ties.—Fig. 10 shows the buttresses and the arrangement and form of the ties, the latter being also shown at *cc* in Fig. 1. The downstream edge of the buttresses is

also seen in Fig. 1 to slope $3\frac{1}{2}$ in. to the foot, while Fig. 10 shows the side batter as 1-40. This gives a top thickness of 22 in. and a base thickness of 5 ft., at which point they are notched into the rock to an average depth of 3 ft., as shown in Fig. 9 at *f*.

The reinforcement of the buttresses consists of vertical 1-in. bars 6 in. inside the downstream face, as at *aa*, while at each side there is a network of $\frac{1}{2}$ -in. bars 2 ft. apart and 3 in. below the surface. On the upstream side are four 1-in. rods at the spring lines disposed in the form of a rectangle and bound with stirrups. Attention is directed to the ties *e* (Fig. 9), which with a thickness of 19 in. have a depth of 2 ft. in the centre section increased to 4 ft. at the ends where they merge into the buttresses. They are reinforced by five $\frac{5}{8}$ -in. bars, *b*, at the top, and three $\frac{1}{2}$ -in. bars, *c*, at the bottom, tied by stirrups at 24-in. centres. The footpath running across the tops of the buttresses is carried by 9-in. beams, as shown at *g*, Fig. 9, the decking, *h*, being 3 in. thick. The panels, posts, and caps of the balustrade are pre-cast. The minor details of what may be classed as subsidiary reinforcement follow on the usual lines of ferro-concrete practice, and do not call for any special notice.

CHAPTER IV

QUANTITY OF WATER TO BE STORED

THE capacity of a reservoir to control the flow of water from a given area of ground used formerly to be calculated on the basis of a proportion of the average rainfall, as two-thirds, one-third being deducted for loss by evaporation and absorption; but it was found by further experience that the loss could not in all cases be taken as any certain proportion of the average rainfall, although in many cases one-third was a near approximation. In the case of reservoirs the capacity of which had been calculated on the basis above named, it was occasionally found that some of the water came down to the reservoir when it was full or nearly so, and passed over the waste-weir beyond control, and, consequently, that instead of the reservoir being full at the commencement of a drought, it was sometimes considerably below the top-water level at such times, and that the quantity stored did not keep up the required daily supply for a sufficient length of time.

The storage capacity of a reservoir large enough to equalise the flow of water over a long drought, or two or three shorter ones with intervening winter rains of less than the usual amount, and so that the annual rainfall is made to yield its daily average quantity throughout long periods of time, can be found when the past rainfall daily for a long time has been ascertained, the number of dry days in the future being assumed to bear the same relation to wet ones as they have done hitherto, and that the past average yearly quantity of rain will continue the same; then the capacity of any reservoir may be found, being measured by the longest period of defect of supply into and discharge out of the reservoir.

The rainfall of a long series of years must be considered in connection with that of a short series of dry years, as three or four. When a table of the rainfall of any locality, extending

over a long series of years, is examined, it is found that the mean depth of three consecutive dry years is about one-sixth less than the general average. The wettest year has a rainfall about half as much again as the general average, the driest year one-third less, and the mean of the driest three consecutive years one-fifth less, than the general average.

Thus in some years a great deal of water passes off the ground by the streams to the sea which cannot be stored. Mr. Hawksley * stated before the Royal Commission on Water Supply, which sat in the year 1868, that gaugings of the actual quantity of water going over the waste-weirs of reservoirs show that it is about one-sixth of the quantity due to the average rainfall, and this agrees with deductions from the tables of rainfall, showing that reservoirs of the capacities of those which permitted one-sixth of the general average rainfall to pass over the waste-weirs do not deal with more than the average depth of three consecutive years of least rainfall. In these cases, therefore, one-sixth of the average must first be deducted, then the loss by evaporation, etc., and the depth remaining will represent the available quantity upon which the capacity of a reservoir may be calculated, according to the number of days' storage required.

A great portion of the water yielded by light rains is absorbed into the ground, and neither flows directly off the ground by streams nor sinks far into it to issue again in springs, but is evaporated, partly from the surface of the ground itself, and partly from the leaves of the vegetation into which it enters, only a small part being left in the vegetation itself, the greater portion of the water which enters into it being evaporated from its multitudinous surfaces.

The annual depth of rain which in this way does not contribute to the streams varies with the nature of the ground on which it falls, being least on a non-absorbent and precipitous area such as mountains of slaty rocks of old geological formation. The longer the time the water takes to reach the reservoir the more of it will be lost, and besides the difference caused by the declivity of the ground the quantity evaporated depends on the humidity or dryness of the air of the locality, and this latter condition varies much in different parts of the country. The depth of rainfall which does not contribute water to the streams at the sites of reservoirs varies from 9 in. to 18 in. in different parts of the country.

* T. Hawksley, Esq., F.R.S., M.Inst.C.E.

The number of days' storage a reservoir for the water-supply of a town should contain varies with the average annual depth of rain and also with the frequency with which it falls. Mr. Bateman, in his evidence before the Commission already named, said that whereas 120 days' storage would be sufficient on the western side of the country and some other parts, where the rain falls on a great number of days in the year, 240 days' storage would be required on the eastern side of the country, where the rainfall is both less in depth and less frequent, and it has been estimated that in certain situations on these sides of the country respectively 150 and 300 days' supply would not be too much.

For the sake of illustration we may take an instance where the average annual rainfall is 42 in. Deducting one-sixth, 35 in. would be the depth to be reckoned upon, and supposing the depth allowed for loss by evaporation, etc., to be 14 in. the available depth would be 21 in. ; and in the absence of exact measurements in every part of the area from which the water from time to time flows, that depth of 21 in. must be supposed to extend over the whole area, if it is not of inordinate extent.

Where the area is very extensive, that depth of rainfall may not extend over the whole of it, and this may have been one of the reasons why expected quantities of water have sometimes not been yielded by a drainage area of given extent ; but it is the nearest approximation to the truth which can be made, and some concession towards it may be considered to be made in deducting one-sixth from the known average rainfall of a few places within the area, although in some cases that quantity has been found to pass actually over the waste-weirs, thus furnishing a direct proof that with such reservoir-room as has been there adopted that part of the water could not be stored.

Runoff.—When large areas are dealt with, the available quantity of water is conveniently reckoned per thousand acres of the watershed area. Upon 1,000 acres an available depth of 21 in. would yield an average daily quantity of

$$\frac{1,000 \times 43.560 \times 1.75}{365} = 208,800 \text{ cu. ft.,}$$

of which about two-thirds might be used for supply to houses, manufactures, or for power away from the stream, or for irrigation, leaving one-third or thereabouts to the stream

itself; or rather, this latter would be first apportioned, leaving two-thirds for other purposes, and the apportionment would stand thus—

One-third = 69,600 cu. ft. per day to the stream.

Two-thirds = 139,200 " " "
 6 $\frac{1}{4}$

870,000 gallons per day for supply, and so on per 1,000 acres, within limits defined by our knowledge of the real average rainfall over any area, the certainty of the calculated average increasing with the number of places of observation over a given area, but becoming too uncertain for use when the area is very large and the places of observation few.

The quantity above set down for the stream would render it more powerful in dry weather than it would be without a reservoir. The average flow of the stream in this case during dry weather would probably not be more than two-thirds of the quantity above stated, for measurements of the dry-weather flow of streams show about 30 cu. ft. per minute per 1,000 acres of the drainage area.

This, no doubt, varies with the nature of the ground from which the water flows, being greater or less as the ground is more or less absorbent. Where the ground in the upper and middle portion of a river-basin consists of chalk, oolite, or sandstone the dry-weather flow forms a larger part of the whole quantity flowing off the ground than where it is of hard and impervious rocks, with the steeper surfaces which always accompany those harder rocks; and the kind of ground which affords a medium dry-weather flow between these extremes is that which consists of alternate formations of sandstone and shale, or of oolite and lias clays, in either case covered in parts with clay and gravel.

The rainfall observations, if sufficiently numerous, will show on an average of years that where the annual quantity is great the number of rainy days is greater than in those parts of the country where the rainfall is comparatively small [this is not, as it might seem to be, a necessary consequence], and so, although more water per day is discharged from a reservoir per 1,000 acres of its drainage area in the former situations than in the latter ones, the loss of water is oftener replenished, and in proportion to the frequency of this action the capacity of the reservoir may be smaller.

The experience in England during the last twenty-five or

thirty years has been that, in order to equalise the flow of water day by day during three or four consecutive years of least rainfall, the capacity of a reservoir should be sufficient to yield about 120 days' supply where the rainfall is frequent, and 240 days' where it is infrequent; but it is here assumed that the reservoir is full at the commencement of a drought, whereas it may not be so if the previous winter's rain has not yielded a surplus over the regular supply sufficient to fill up the reservoir completely, and in this view the capacity should be increased perhaps to 150 days' supply in the one case and 200 days' in the other.

From an examination of Mr. Symons's * annual records of rainfall it appears often that nearly as many rainy days occur in the southern and eastern counties as in the western and midland counties, although the rainfall in these is much greater; but the days set down as rainy in the records include all those on which one-hundredth of an inch of rain has fallen, and this is so little that it often adds nothing to the run of the streams. The rainy days, the frequency of which affects the capacity of storage reservoirs, are not numerous.

If in the example where 21 in. is the available annual depth of water over the initial area of 1,000 acres, the proper number of days' supply were, from its position, say 180, the capacity of the reservoir would be $208,000 \times 180 = 37,584,000$ cu. ft. Deducting the dry-weather flow due to 1,000 acres at the rate of 30 cu. ft. per minute, that is 7,776,000, there remains 29,808,000, or say 30 million cubic feet, as the capacity of the reservoir, which is at the rate of 30,000 cu. ft. per acre of the watershed area.

Whether the dry-weather flow of the drainage area should be deducted from the capacity of a reservoir in the manner already stated, will depend upon the method by which the number of days' storage is arrived at; whether upon the basis of (1) the rainfall table, showing the absolute length of time during which no rain has fallen; or (2) upon the stream gaugings, showing the daily quantity reaching the site of the reservoir; or (3) upon the difference between two quantities, one of which is the quantity of water in an existing reservoir at the commencement of a drought, to which is to be added the quantity coming into the reservoir during the drought, whether from springs or casual rainfall—and the other quantity is that which has been discharged during the drought,

* G. J. Symons, Esq., F.R.S., F.R.Met.Soc.

as ascertained by the daily sinkings of the water-level of the reservoir, in which case it is evident that the total quantity is dealt with on either hand, and therefore no deduction can properly be made.

But this method is true only of that reservoir, unless the dry-weather flow, where it is proposed to make a new one, is the same, area for area; whereas by the other method the rule becomes general, and is applicable alike to cases where the dry-weather flow is large, comparatively, and where it is small. It is therefore better as a method, although the other is more exact for a particular case already existing. The dry-weather flow of the stream at the site of the reservoir is made up of the numerous small springs issuing within the drainage area, produced because of the absorbency of the ground during wet weather, and its yielding of the absorbed water gradually, and to this extent of the dry-weather production the drainage ground is essentially a part of the reservoir, and for this reason the quantity should be deducted from what would otherwise be the necessary capacity.

Were the ground wholly impermeable there would be no run of water in dry weather, and, in that case, the capacity of the reservoir itself would need to be so much larger; but where, as is assumed in the example, there is a dry-weather flow which, on the average, during a drought, is equal to 30 cu. ft. per minute per 1,000 acres, that quantity may be deducted from the calculated storage-room in order to arrive at the actual capacity of the reservoir. The quantity so deducted belongs to the reservoir, and is, in fact, part of the yield reckoned upon in the available depth assumed, as, in the case adduced, 21 in., and whether it be conducted past the reservoir and form part of the supply, or it run through the reservoir and form part of the discharge into the stream, makes no difference in the reservoir-room proper to be provided; but where the number of days' supply to be stored is founded on the length of a drought as ascertained from the rainfall returns, the dry-weather flow ought to be deducted. But the formation of the ground on reservoir sites varies much, so that any rule of this kind gives only an approximation to results in particular cases.

The height of the top of the embankment above top-water level is often 4 ft. in small reservoirs and 6 ft. in large ones. Where the depth of water at the site of the embankment does not exceed about 30 ft., the top of the bank may

usually be 4 ft. above top-water, or the level of the waste-weir. With a depth of from 30 ft. to 50 ft., the top of the bank may be 5 ft. above that level, and where the depth is 60 ft. and upwards, the height should be 6 ft. ; but it somewhat depends upon the direction in which the reservoir lies lengthwise, and to its exposure to gales of wind. Irrespective of this, a certain height is necessary to meet a flood which may raise the water-level above the waste-weir, and as a matter of prudence and safety the waste-weir should be of such length as to prevent the water rising, on the occurrence of the greatest flood, to a height more than about 2 ft. above the waste-weir. But beyond that, an allowance must be made for the further height to which the water may be driven by wind, and the greater the depth of water at the embankment the longer will the reservoir be on the same site, and on a long reservoir the water is driven up by the wind to a greater height than on a short one.

In the upper parts of river-basins where reservoirs may be made, the opposite hill-sides approach each other towards the bottom, and almost meet in the stream with straight slopes. This is a general characteristic ; but a really good site for the formation of a storage reservoir widens out in the bottom above the site of the embankment, and the only way of ascertaining the quantity of water the site is capable of affording, with any given height of bank, is by actual measurement, either by sections across the valley or by contour lines laid down upon a plan every 4 ft. or 5 ft. in height ; but we may estimate approximately what quantity of water a fairly good reservoir site would probably afford by examining sites the capacities of which have been ascertained, and comparing these with the respective heights of the embankments, or, rather, with the greatest depths of water, D , and their areas, A , taking these also at several different heights on the same site. The results of an examination of 75 such cases where the depth D varies from 20 ft. to 80 ft., both inclusive, are as follows : In each case the cubic capacity is divided by the area of the water surface, giving the average depth ; this then is divided by the greatest depth, D , giving the ratio set down opposite each case.

The general average of these 75 cases is about $\frac{4}{9}D$. The shape of the ground, as it widens out above the site of the embankment, makes the average width of the reservoir in all

these cases about twice as much as it would be with the straight slopes assumed to meet in the stream at the bottom of the valley, in which case the width of the reservoir is greatest at the embankment ; whereas, in the better sites the

No.	Ratio.	No.	Ratio.	No.	Ratio.	No.	Ratio.
1	.612	20	.49	39	.452	58	.4
2	.583	21	.487	40	.45	59	.4
3	.580	22	.485	41	.45	60	.4
4	.572	23	.473	42	.447	61	.4
5	.560	24	.468	43	.435	62	.4
6	.543	25	.468	44	.431	63	.396
7	.537	26	.468	45	.427	64	.395
8	.528	27	.466	46	.422	65	.394
9	.526	28	.466	47	.42	66	.38
10	.525	29	.465	48	.42	67	.376
11	.515	30	.464	49	.42	68	.35
12	.515	31	.463	50	.42	69	.35
13	.514	32	.462	51	.42	70	.345
14	.510	33	.46	52	.42	71	.34
15	.504	34	.46	53	.417	72	.34
16	.5	35	.46	54	.407	73	.336
17	.5	36	.46	55	.407	74	.32
18	.496	37	.458	56	.405	75	.281
19	.496	38	.455	57	.4		

greatest width is considerably above the embankment, the width there being nearly the average width of the whole reservoir. In the case adduced the area is $3,000 \times 450 = 1,350,000$ sq. ft., and the greatest depth being 45 ft., the capacity of the reservoir is

$$A \times \frac{4}{9} D = 27 \text{ million cubic feet.}$$

CHAPTER V

APPURTENANCES OF RESERVOIRS

A RESERVOIR for a town supply, as distinguished from a compensation reservoir, sometimes has a bye-channel excavated along the margin of the top-water level from the head of the reservoir to the bye-wash which takes the water from the waste-weir. It is useful for several purposes, even when not absolutely necessary. To begin with, it furnishes a considerable quantity of materials for the bank ; it enables the bottom of the reservoir to be laid dry for any purpose ; and if a discoloured flood should come down when the reservoir is full, or nearly so, it affords a means of excluding the objectionable water, if made large enough. Where a flood channel is not absolutely necessary, these considerations ought to weigh in favour of a bye-channel being constructed.

At the head of a reservoir it is a good arrangement to have a short bank across the stream for the purpose of arresting sand, stones, alluvium, or what else may be brought down in floods, forming a receptacle known in Lancashire as a lodge, in connection with which the flood-weir and sluices are constructed.

Where it is desirable to separate the clear spring waters of a drainage area from the flood waters, an arrangement, depending for its action on the laws of water in motion, which was first introduced by Mr. Bateman, the eminent engineer, may be applied. It consists of a side channel introduced under and across the line of watercourse, into which the water that arrives at the edge of a weir above it without violent motion drops easily down and is carried by the side channel away from the reservoir, while on the arrival of a flood it overleaps the narrow space down which the purer water drops, and flows over a second weir and away to the flood-water reservoir.

In exposed situations—and such reservoirs as are here

described are usually in such situations—the wind acting on the surface of the water surges it against the face of the embankment in such a manner that without some protection the earthwork would be gradually washed away. To prevent this, it is necessary to pitch the face with a harder material, usually stone, to the depth of from 12 in. to 18 in. Flat-bedded rubble stone makes good work, but where the slope is, as it should be, as flat as 3 to 1, rough rubble spread over it stands well enough where the bank is not exposed to high winds.

It is always desirable to know what quantity of water is flowing out of a reservoir, and in some cases, viz., where mills are compensated by a stated quantity per time measurement, it is necessary that the water should be gauged either by allowing it to flow over a weir, or through an opening below the head of water; but the gauging is perhaps, on the whole, more accurately and satisfactorily made over a weir. To bring the water to a sufficient state of rest before it flows over the weir, it is first received into a basin, out of which it flows quietly.

Sometimes a flood channel is not made, but the water coming into the reservoir during its construction is got rid of through the discharging pipes or culverts, and if more than they will discharge comes down and seems likely to reach and go over the partly-raised bank, temporary shoots are used to convey it over to the outside; and when no flood channel is made, the waste-weir becomes an important feature of safety, or otherwise, according as its length is properly proportioned to the drainage area or not.

Spillways.—No discharge pipes or culverts of practicable dimensions would carry off such a flood as three or four hundred cubic feet of water per second from each 1,000 acres, and yet such a quantity must be expected to come into the reservoir some time or other when it is full. It is plain, therefore, that even with the utmost watchfulness of the reservoir-keeper in opening the sluices on the approach of a flood into a full reservoir, some other and larger means for its escape must be provided, and for that purpose a waste-weir is formed in the solid ground at one end of the embankment, with a bye-wash to carry the water safely to the stream below. If we consider that with a height of top-bank 6 ft. above top water, or the level of the weir, 4 ft. of that space would be little enough to oppose to the wash of water produced by a gale of wind

blowing down the reservoir, we shall see that it will be very undesirable, and even unsafe, to allow the water to rise to a greater depth on the weir than about 2 ft.

Taking that depth, then, as the greatest to be allowed, and 400 cu. ft. per second per 1,000 acres to be the greatest flood, and 100 cu. ft. per second to be the greatest quantity that the discharge pipes or culvert would deliver, leaving 300 cu. ft. per second to go over the waste-weir, it will be seen that the weir would require to be of a length of 37 ft. per 1,000 acres of the drainage area. Here it is assumed that 4 would be the proper coefficient in the equation $q=c\sqrt{d^3}$, d being in

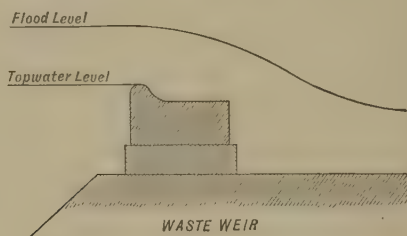


Fig. 12.

inches, for such a depth over such a weir, which would, from its breadth, if built of ashlar, reduce the coefficient to about that number if the depth were only 1 ft. ; and, seeing that the greater the depth the more the weir loses its property of a free fall over, or becomes drowned, it may well be supposed that, if it could be experimentally determined for such a depth as 24 in., as it has been for depths up to 12 in., the coefficient would be found not to be higher than 4. Using that coefficient, then, and limiting the depth over the weir to 2 ft., it will be seen that 37 ft. or 38 ft. length of weir is necessary for every 1,000 acres of the drainage area.

This does seem a great length, and it is certainly greater than most waste-weirs possess, but with a shorter weir there might be a risk of the water rising too near the top of the bank, when that is not more than 6 ft. above the top-water level. If questions of economy could be allowed to enter as freely into the design of waterworks reservoirs as they do into that of many other works, an engineer might contrast the improbability of an excessive flood occurring at the very time of a full reservoir, and might conclude to save the expense

of a long weir, or to allow the water to rise to a greater height upon it than 2 ft. or so ; but, all things considered, he would be a bold man who would allow the water to rise upon the weir to a height more than half the height of the bank above it, where that height is 6 ft., and if the water were allowed to rise to the extent of 3 ft. on the weir the length per 1,000 acres would require to be 20 ft., for a flood over the weir amounting to 300 cu. ft. per second per 1,000 acres of the drainage area.

An instance in which something like this proportion was adopted is the Grimwith compensation reservoir of the Bradford Waterworks. Here the drainage area is about 7,000 acres, and when the late Sir William Cubitt was referred to to fix the length of the waste-weir of that reservoir—being the person agreed upon mutually by the mill-owners and the waterworks proprietors to fix the length of the weir—he prescribed a length of 150 ft. This is in strong contrast to the length of another weir in a situation not very dissimilar, and where the drainage area is also nearly 7,000 acres, viz., at Tittesworth, in North Staffordshire, a compensation reservoir belonging to the Staffordshire Potteries Waterworks, where the waste-weir for this very large area was made no more than 60 ft. in length, the height of the top of the bank being 6 ft. above the level of the weir. The consequence of making the length so short was, that in August of the year 1862 the water rose to a height of 5 ft. above the weir ; and if only the accidental occurrence of a wind blowing down the reservoir had taken place at the same time, there can be no doubt that a great disaster would there and then have happened.

In the case of the Bradfield Reservoir, a compensation reservoir belonging to the Sheffield Waterworks, which burst in 1864, it had a drainage area of 4,300 acres and a waste-weir of 60 ft., in a horseshoe form ; and although the bursting of the embankment was due to other causes than the shortness of the waste-weir, there can be no doubt that if the bank had been otherwise soundly constructed upon ground which could not move, its failure would at some time have been imminent from this cause alone.

Although, perhaps, in the greater number of cases the waste-weir is made at one end of the bank, in the solid ground, it has been the practice of some engineers to combine it with the structure necessary to be erected at the inner end of the discharge pipe or culvert, for the purpose of working the valves. This structure consists of an enlarged valve well,

of such diameter that the circumference of its top course shall be of sufficient length for a waste-weir. This valve pit affords a ready and convenient access to the inner valve of the discharge pipe, and it admits also of the facility of drawing off

the water at different heights, for the water near the top is generally better than that drawn from the bottom, and for this drawing-off purpose valve pits are sometimes built, although they are not made use of as waste-weirs.

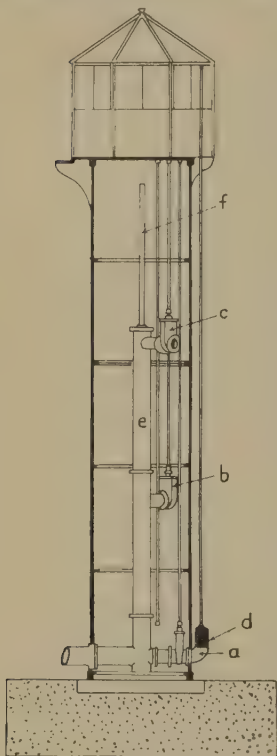


Fig. 13.

Valve Towers.—A usual feature of the storage reservoir is the valve tower. Approached from the bank by a light steel footbridge, this tower is generally built up from the deepest part of the reservoir. From it the main outlet pipe is taken either through or around the end of the bank as previously suggested. Sometimes these towers are built of masonry or concrete, but for moderate-sized works, the cast-iron tower as shown in Fig. 13 is a suitable arrangement, and enables the structure to be erected with the minimum of trouble. In this particular instance it will be seen that three draw-offs are provided for at *a*, *b*, and *c*, each of which is provided with a copper-gauze screen *d*. They are all connected to the central stand-pipe *e* and have valves as shown. A depth indicator may be attached at *f*. The tower, it will be observed,

is constructed of circular cast-iron sections with machined flanges, while the valves are operated from the top platform, to which also the screens may be drawn up for cleaning.

CHAPTER VI

DISCHARGE OF WATER FROM THE RESERVOIR

THERE are two distinct principles involved in the practice of drawing off the water from a reservoir ; the one is to discharge it through pipes laid so securely and so guardedly as to form a feature of stability equal to that of any other part of the reservoir—as the puddle wall, for instance, or the bank itself—thus rendering unnecessary any provision for inspection or repair, in the same way that one has to trust to the stability of construction of the puddle wall, where inspection is impossible. This mode of discharging the water was frequently adopted some years ago, but of late years not so much, if at all ; it consists of one or more lines of cast-iron pipes laid in the solid ground underneath the embankment from the foot of the inner slope to that of the outer one, with valves on them to control the discharge of water. Sometimes these valves are placed at the inner ends of the lines of pipes, and sometimes at the outer ends ; the former position being the more secure and the latter the more convenient. Mere convenience, however, in reservoir works, cannot be much considered.

When the valves were placed solely at the outer ends of the discharging pipes they were in duplicate, so that in case of one being out of working order the other might be used. When valves were so placed the mouth of the pipe within the reservoir had sometimes a conical plug suspended over it, so that it might be dropped into the mouth in case of emergency. With large pipes or great heads of water, the pressure on this plug would prevent its being raised again without very great power, if it were not that the main plug contains in its crown a smaller one, which, when it is desired to raise the plug again after use, is first lifted, and, the valve at the outer end of the pipe being closed, the pipe is filled with water and the pressure on the large plug equalised, after which it is easily raised.

There are cases where this precaution of either valve or

plug at the inner end of a line of discharge pipe is not taken, but these are cases of the boldest and most self-confident treatment, and verge, indeed, on the indiscreet; and anything even approaching indiscretion in the construction of reservoirs is certainly to be discouraged.

On this principle of drawing off the water the pipes are made unusually strong, the sockets unusually deep, the depth of lead unusually great, and everything, in fact, is done to render the pipe secure under its circumstances. On the other principle the pipe is not made a special feature of stability, but is enclosed in a culvert of masonry of such a size as to admit of inspection and repair of the pipe. It is quite evident that either pipe or culvert should be laid on an unyielding foundation. To admit the possibility of sinking in either case would be absurd. There have been a few cases where pipes have been carelessly laid in this respect, but they have been condemned as bad construction.

If we take first the principle of laying a bare pipe under the bank, it will be seen that an element of weakness presents itself in the liability of the water to follow the line of the pipe between its surface and the material adjoining it. To prevent this it is enclosed in clay puddle, and obstructions to this action of the water are placed at right angles round the pipe. The rims of the sockets themselves form a considerable obstruction to the passage of water, but an additional precaution is sometimes taken to place wider flanges, projecting, say, 6 in. all round the pipe.

The discharging pipes are laid not from the lowest level of the reservoir, but at a level some feet above the bottom, and it is of advantage to lay another pipe from the bottom itself, which shall have its outlet in the stream below, so that the reservoir may be completely emptied when necessary.

Crossing the puddle trench is the great difficulty of the line of discharge pipe. The puddle will not carry weight, and the pipe has to be made to carry itself and all above it from side to side of the puddle trench, unless it be very wide, when a solid pier of masonry is brought up from the bottom of the trench in the centre, to afford a central bearing to the pipe. The transverse strength of this pipe is capable of estimation as well as any other cast-iron girder, for it becomes a cylindrical cast-iron girder, of such strength as to carry the load due to it, with, of course, in such a situation, a very large margin for contingencies.

Mr. Bateman directed this difficulty to be met, in the case of the Halifax Waterworks, by bringing up a solid ashlar pier of the full width of the puddle trench, dressing down the sides of the trench where the masonry abuts against them, and filling the joint with cement. In the middle of this pier, lengthwise of the trench, an iron plate is brought up, projecting 12 in. beyond each face into the puddle, and standing up 18 in. above the pipe, the joints between the iron plate and the masonry being filled in with cement, and other precautions were taken in passing the discharge pipe through the iron plate.

There are not wanting on this subject, as on some others, differences of opinion as to the desirability of placing bare pipes under embankments. Culverts of masonry, it is said, are preferable, through which the water may be discharged either openly or through a pipe which can be got at for examination and repairs, and there are some very successful examples of this mode of construction on the Bradford Waterworks, designed and carried out by the late Mr. John W. Leather, of Leeds; as good, perhaps, in design and execution as any in the country.

After the failure of the embankment of the Bradfield Reservoir of Sheffield, where the discharge pipes had been laid on the principle of trusting to the pipe, but where several precautions usually considered necessary had been omitted, it was very much the opinion of Mr. (now Sir) Robert Rawlinson, who, with Mr. Beardmore, investigated the case on the part of the Government, that the bank failed because of defects in the discharging pipes, and Mr. Rawlinson went so far, in some general remarks on reservoirs, as to condemn *in toto* the principle of laying a bare pipe under an embankment, and urged strongly the necessity of enclosing the pipes in culverts of masonry large enough to admit of workmen passing up them.

One ought, however, to consider whether the larger culvert does not invite strains, requiring enormous resistance, which the comparatively small pipe does not, and which is therefore so far preferable.

The joints of a large culvert are no doubt a source of weakness, and it is to this point chiefly, and to that of the possibility of crushing the materials, that attention is drawn when considering whether a large culvert or a small pipe is preferable.

If there are to be two discharges from the same reservoir—viz. one for supply and the other for compensation to the stream—the supply pipe would be laid through the discharge culvert, up to the valve tower at its inner end, whether it be situated in the reservoir, clear of the embankment, or built within the embankment itself. The former is the better plan. The best position of the discharge pipe or culvert, or the line along which it should be laid, demands the most serious consideration in every case, in order to hit the happy medium which combines security with economy.

The level at which it must necessarily be laid being near the bottom of the reservoir, and the puddle trench being almost as necessarily sunk much below that level, the pipe or culvert, when laid near the middle of the bank, must cross the puddle trench at a considerable height above the bottom, and right through the mass of puddle. This is the plan which was formerly adopted, whether the discharge pipe was enclosed in a culvert of masonry or was laid bare in the ground; but as it was necessary to support the pipe or culvert across the trench upon a more solid foundation than the puddle itself afforded, the unequal settlements which took place when the bank had been made to its full height tended to fracture the work.

To obviate this, straight vertical joints completely across the whole structure of the culvert were sometimes provided at the points where the unequal settlements would be likely to occur—slip joints—so that the portion of the culvert between the slip joints might settle bodily and evenly. In the same way the brickwork of railway and other tunnels driven through yielding ground is made with a straight joint at the end of every length, instead of leaving toothing for the next length, which is proper enough, and more sightly when finished, where the ground is in unyielding rock.

This provision of a straight joint, however, in the case of reservoir embankments, has sometimes not had the expected effect, owing to the uncertainty of the direction in which the settlements take place, there being horizontal as well as vertical movements of the earth above the culvert. The culvert, therefore, should not cross the puddle trench at any great height, if at all, above the bottom of the trench. This consideration drives the position a long way from the middle of the bank, in order to lay the culvert in the solid ground.

The bottom of the reservoir, to a height of 14 ft. or 15 ft.

above the lowest point, contains but little water—say in this case half a million cubic feet. It is not necessary, therefore, to lay the discharge pipe lower than is sufficient to draw off the water to this level, say 40 ft. below the top-water level, and the lowest sluice should be, say, 3 ft. below this level, to give a head of entrance into the pipe, thus fixing the level of the upper end of the discharge pipe at 43 ft. below the top-water level of the reservoir, by which means 46,000,000 cu. ft. of water can be drawn off from a full reservoir. If the invert of the culvert be laid 2 ft. lower than the pipe, it would form a channel for the compensation water to be discharged into the stream. To determine the size of the culvert, that of the supply-pipe must first be found. The daily quantity of water due to 1 in. rainfall on 2,000 acres of ground is, as before found, 258,570 cu. ft.

If one-third be discharged into the stream as compensation, the quantity passing through the pipe would be 172,380 cu. ft. per day. If the mean daily velocity through the pipe be made 2 ft. per second, the diameter found from these data would be 15 in. In assigning room for the 15-in. pipe, which would be laid before the arch of the culvert is turned over it, it should be considered that it might be possible at some future time that it would be necessary to cut out, remove, and replace one of the pipes, in which case more room would be required than is necessary for laying the pipe in the first instance; the width and height, therefore, should be sufficient for this—say 5 ft. wide, and $5\frac{1}{2}$ ft. high.

The width of the trench for the culvert would be about 10 ft., and it should be made straight as far as it extends under the embankment, and some distance away from the foot of each slope, as shown from A to B in Fig. 2; but excavation may be saved towards the ends by inclining them inwards from the points A and B to C and D respectively.

In laying the culvert in the trench thus excavated in the solid ground at such a depth below the head of water, every possible care is to be taken to prevent the water finding a way outside it, and for this purpose gravel puddle is a suitable material with which to solidly fill every space not completely occupied by the masonry. It may be thought that concrete is a more suitable material for this purpose, and in any case the invert of the culvert would be laid on a bed of concrete, unless the ground consist of a solid rock; but for filling in the spaces between the back of the masonry and the sides of

44 DISCHARGE OF WATER FROM THE RESERVOIR

the trench gravel puddle is a better material, inasmuch as it can be made more compact and watertight in such a situation.

The valve-tower at the upper end of the discharge culvert may be of brickwork 18 in. thick, 6 ft. diameter inside. The lowest valve being 43 ft. below the top-water level, there might be three more above it, at 30 ft., 20 ft., and 10 ft. below that level, so that the water can be drawn off from near the surface at all times. These would all communicate with one descending pipe erected within the valve-tower, from the top of which they would be worked, a light iron footbridge affording access from the top of the bank. Side by side with the lowest valve would be another for drawing off the compensation water. This should be of larger size than the supply-pipe; in this case it might be 2 ft. diameter, if circular, but a rectangular form is preferable.

The inside slope of the embankment would be covered with rough stones, the interstices being filled with smaller broken stone, forming a protection to the earth from the wash of water. The area to be so covered would be about 6,000 sq. yds. The outer slope, to be soiled, would be about 4,000 sq. yds., and, including a grass border on each side of the road to be formed on the top of the embankment, the quantity would be 4,800 sq. yds.

A more important part of the work is the waste-weir and bye-wash. For such a drainage area as that assumed the length should not be less than 60 ft. The bye-wash may be nearly wholly of concrete, and by forming the bye-wash in a series of ponds, of not more than 6 ft. difference of level between them, the descent of the water, in the heaviest flood, may be sufficiently checked, and it may be gradually let down to the stream below the embankment without injury.

A large body of water may be dealt with safely if it be prevented acquiring a too great velocity. Each of the steps shown in the sketch forms an independent pond into which the water falls from a moderate height. The general inclination is, in the example, 1 in 8, each step 6 ft., and the length from step to step nearly 48 ft.

There would be, in a flood, a great body of water, but no great velocity in any part of the channel; and, except for the overfall sills, which should be of stone, the whole of the bye-wash may safely be formed of concrete. If the width at the level of each overfall sill be made 30 ft., or half the length of the weir, when the water is flowing 2 ft. deep over the weir, the corresponding depth over the sills would be 3.17 ft.

CHAPTER VII

STREAM GAUGES

THE watershed line along the tops of hills sheds the rainfall in opposite directions into different valleys—the basin, when taken as a whole, in each case, from source to sea ; or the catchment area, or drainage area, or watershed area, when a part only is dealt with.

Of the rain falling upon this watershed area and running off in streams, some is lost by evaporation in the atmosphere and absorption into the ground before it arrives at the point down to which the area is calculated, and the quantity lost can only be ascertained by gauging the flow of the streams and comparing the quantity so found with the whole quantity due to the area and depth of rainfall. The quantity lost varies with the declivity and character of the ground.

After a sufficient number of gaugings of the streams has been made, the practical quantity which can be depended upon may be ascertained. Such gauges, being of a temporary nature, may be made with planks, the joints being grooved and tongued, and the whole bolted together and stiffened with battens. A stream should be gauged daily at least once, but that is not sufficiently often for accuracy, for within twenty-four hours the depth of water flowing over the gauge may vary greatly. Twice a day, or three times, is not too often—say at 6 a.m., 1 p.m., and 8 p.m.—and even these frequent observations need to be supplemented by notes of the beginning and end of rains sufficiently heavy to influence the stream of water ; but these are not so necessary if the twenty-four hours be divided into three equal periods of time for the observations of the depth.

In any gauge of this sort, whether temporary or permanent, it is necessary to dam up the stream of water and form a considerable pond in which the water may expand itself and flow out quietly over the gauge, and from as nearly still

water as possible. If this precaution be not taken the velocity of the water before arriving at the gauge would have to be measured at the time of every observation, and this would admit numerous possibilities of error.

Hydraulicians have measured the actual quantities of water flowing over weirs and notches in thin plates and plank-gauges, by receiving into a large tank below the gauge all the water flowing over it in a given time at various depths, every condition of the flow of water and the construction of the gauge being observed at the same time—such as the width of the gauge with reference to the width of the pond ; the thickness of the lip or sill over which the water flows ; whether the edge be horizontal, crosswise of the gauge, or bevelled off, and if bevelled, which direction, whether outside or inside ; and the same with respect to the ends of the openings ; the velocity of the current (if any) before arriving at the gauge ; and whether the depth recorded be that from the surface of the water in the pond or that directly over the sill of the weir or in the notch ; because this latter is always considerably less than the former, and requires the application of a different rule, and not one of these particulars can be omitted to be recorded if the gaugings are to have the value which the labour bestowed upon them would otherwise give them.

The depths have been mostly measured from still water, and the results have been formulated accordingly ; but it is not sufficient to assume that this has always been the depth measured, unless it has been expressly stated so. Some of the experiments have been made over thin plates of copper or iron, some over thicker plates of iron, some over boards of various thicknesses, and as these and other particulars have varied, the actual quantities passing into the outer tanks have varied for the same depths ; and in adopting a thickness of the lip of the gauge, regard should be had to the conditions of the experiments relied upon, so as to place the gauge under similar conditions to those of one set or another of these.

Almost any set of experiments may be relied upon if the conditions under which they were made be duly observed ; but if they be taken together indiscriminately they form nothing better than a muddle of facts from which nothing certain can be determined. For this reason, in fixing a temporary plank-gauge, for instance, where the water can be dammed up into a pond large enough to prevent it reaching

the gauge with any distinct current, 2 in. is a better thickness than 3 in., because a greater number of accurate experiments has been made over planks 2 in. thick than over 3 in. planks.

And with reference to other experiments, the same precaution should be taken. Mr. Blackwell's * experiments were made over various thicknesses of lip and widths of sill, thin plates, 2-in. planks, and wide-crested weirs, and over various lengths of gauge with the same depth of water,—3 ft., 6 ft. and 10 ft. Those over 2-in. planks were 120 or more in number, from 1 in. to 14 in. in depth.

Those experiments of Mr. Blackwell, above referred to, were made on the Kennet and Avon Canal, and were made from a still head of water ; but it will often happen in gauging streams that a pond large enough to effect this cannot be made, and in that case a set of experiments—also over a gauge 2 in. thick, of iron—made at one of the reservoirs of the Bristol Waterworks, more resemble the conditions of temporary stream gauges, the pond being not so large, and the water arriving at the gauge with some current, though not much. The coefficients found from these experiments were therefore greater than those found from the experiments first referred to.

This latter set of experiments was made by Mr. Blackwell in conjunction with the late Mr. James Simpson, the engineer of the Bristol Waterworks, who furnished to him a set of observations he had previously made on the same weir, which is the weir of the gauge-basin of the Chew Magna Reservoir, and is 10 ft. long. The experiments were about 60 in number, from 1 in. to 9 in. in depth.

If it is intended to rely upon those experiments which have been made over thin plates, the number of which is, perhaps, greater than has been made with planks of any thickness, although most of those over thin plates have been on a smaller scale, a strip of sheet-iron may be screwed on, on the upper side of a plank-gauge ; but in that case it should project clearly beyond the edge of the plank, so that the edge of the plank may have no influence upon the flow of water, and when a strip of iron is thus attached on the upper side of the opening, the edges of the plank should be bevelled outwards, away from the iron plate.

With reference to the dimensions of a gauge, regard must be had to the area of the drainage ground and to the probable

* The late T. E. Blackwell, M.Inst.C.E.

variation in the rate of flow of the water. The gauge must allow floods to pass through it or over it, and at the same time it must be small enough in width to gauge the dry-weather flow. The one quantity may be a thousand times more than the other.

The variations being so great, it is necessary to confine the smaller quantities to a less width than that of the full gauge, by making a notch in the gauge of such limited width as will afford a measurable depth, and one which will accord nearly with some of the depths which have been experimented upon.

The width of the notch, as well as that of the whole gauge, should be apportioned in some degree to the area of ground from which the water flows. To take, for an instance, 300 acres, there may flow from it as small a quantity as 10 cu. ft. per minute, or less, and a flood may amount to 9,000 or 10,000 cu. ft. per minute. If the notch be made 2 ft. wide, 10 cu. ft. per minute would pass through it with a depth of 1 in., or thereabouts; and if the depth of the notch be 6 in., the quantity would be about 140 cu. ft. per minute when running full. A considerable rainfall would then have taken place. Greater quantities of water than this would extend over the whole width of the gauge.

To avoid errors, it is well not to subdivide the width, and form more than one notch. If the full width be made 6 ft., the gauge would pass 3,000 cu. ft. per minute when running 1 ft. 9 in. deep, and this would be deep enough for the full depth of the gauge, in consideration of its strength.

Floods greater than 10 cu. ft. per minute per acre of the drainage ground would pass over the top of the gauge, and could not, perhaps, be accurately measured; but the gauge ought to be so set that a proper record of the depth can be made, and some tolerably approximate estimate made of even the greatest flood.

Construction of Gauges.—To resist floods, the gauge must be secured in masonry walls at the ends, the ends of the plank-gauge being built into them, or by piling, according to the nature of the ground. The first thing to be done in fixing the gauge is to divert the water from its channel, if possible; but if not, to divert it from each half alternately, by a clay dam, and to pump out the water and excavate a trench across the bed of the stream and well into the banks on each side, of a width of about 3 ft. and about the same depth, to be filled with puddled clay; and when a depth of 2 ft. of puddle

has been formed in the bottom, the gauge should be set level upon it, and the filling of the trench on the upper side of the gauge completed up to the bed of the stream.

On the lower side the puddle will be covered with the pitching-stones of the apron, the top of which should be at

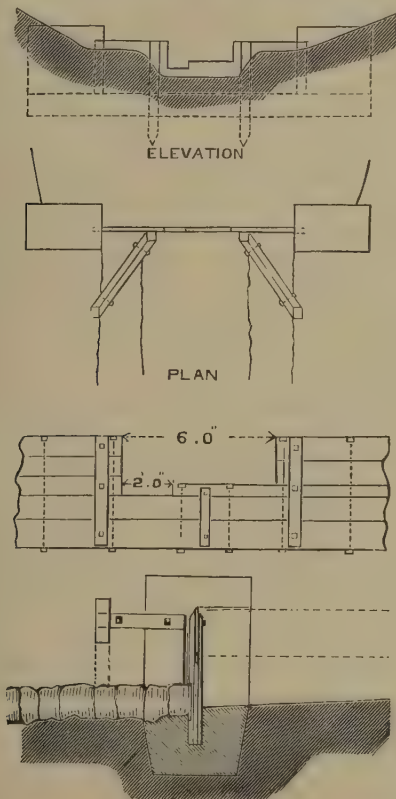


Fig. 14.

least 6 in. below the notch. If the pitching-stones are 18 in. deep, this makes the depth of the gauge below the notch 2 ft. If the depth of the notch be 6 in., and the top of the gauge 1 ft. 9 in. above the top of it, the whole depth of the gauge at the ends will be 4 ft. 3 in. The length of the gauge would be 20 ft., the faces of the abutment walls between which it is

fixed being 18 ft. apart, and each end of the gauge should be built 1 ft. into the walls. Such a gauge might be made of 3-in. planks, the edges being reduced to 2 in., unless thin iron plates be attached. It would require to be strongly battened, and it should be supported by piles, 12 by 12, in the positions shown, strutted from other piles in the banks of the stream. To allow the greatest flood to pass over the top of the gauge with a depth not exceeding 15 in., the total length between the walls would require to be 18 ft., and the walls should be carried up to that height above the top of the gauge, and as far into the banks as to prevent the passage of water round them.

It must not be overlooked that the depth of the puddle-trench named, 3 ft. below the bed of the stream, is but little, considering the height of the water above it in a flood, which might be $4\frac{1}{2}$ ft. or 5 ft., and it is possible that the water might blow a hole through the ground under the puddle. It is a contingency to be considered, but it is not a thing which would probably occur, and the apprehension of it need not prevent a gauge being fixed in the manner described. At the same time, if circumstances are favourable, it might be better to carry the puddle-trench deeper.

In the case of a very loose bottom of sand and gravel—but not in a compacted gravelly-clay bottom—a row of sheet-piling would be preferable to a puddle-trench, for which 3-in. deals would be sufficient, and if the bottom is silt, wholly or in great part, piling becomes a necessity, as in such ground the water would be very likely to flow out under the puddle. It is important that the gauge not only be set level from end to end, but that it remain so, and a gauge of this length should be supported by two intermediate bearing piles, or large stakes, driven into the bed of the stream, 4 in. to 6 in. square, or 5 in. to 7 in. diameter; and these intermediate supports are equally necessary, whatever the ground may be.

The dimensions of the gauge shown in the diagrams are suitable for an area of 300 acres or thereabouts, and for larger areas it will be preferable to gauge the quantity of water at two or more different points of the area if more than one stream proceed from it, rather than, for a temporary gauge, to place one to command the whole area; for a plank-gauge cannot well be made longer than about 20 ft., or wider than 4 ft. or 5 ft.

Below the bottom of the notch 2 ft. is little enough, while 2 ft. above the top of it is high enough, in consideration of

its strength to resist a flood, and, indeed, this it would not do, with these dimensions, unless firmly supported laterally near the middle, either by struts from the banks or by raking struts from piles driven into the bed of the stream, or by raking walls built in the same positions. The planking, indeed, is to be looked upon as being merely a watertight shield, without lateral strength.

The width of the pond immediately above the gauge should be considerably wider than the total length of the gauge—in this case not less than 30 ft.—and the depth of water should be measured at a distance of 6 ft. above the gauge, for which purpose a stake should be driven as far from the edge of the pond as will allow the divisions of the scale to be distinctly read, and on the stake a scale divided into hundredths of a foot, or into tenths of an inch, should be fixed with its zero level with the bottom of the notch.

Formulæ.—The depth of the water passing over the gauge-weir affects the quantity in two respects; first, as being directly proportionate to the quantity, as the width is, the two dimensions forming the sectional area calculated upon; and, secondly, it affects the quantity as being a measure of the velocity with which the water passes over, this being proportionate to the square root of the depth.

Dividing the cubic quantities measured in a tank, which have passed over the gauge-weir at known depths, by all the dimensions before stated, numbers result which are the constant multipliers required to convert the general formula to practical use.

If c = the constant multiplier.

w = the width of the gauge in feet.

d = the depth of water in inches.

Q = the quantity in cubic feet per minute.

$$Q = c w d \sqrt{d} = c w \sqrt{d^3}$$

The values of c , derived from Mr. Blackwell's experiments before mentioned, made on the Kennet and Avon Canal, are as follow for 2-in. plank weirs, and for iron plates $\frac{1}{16}$ in. thick.

Depths,	Values of c .					
	2-in. planks.			Thin plates.		
1 in.	..	3.50	..	..	5.72	
2 in.	..	4.25	..	..	5.70	
3 in.	..	4.44	..	..	4.91	
4 in.	..	4.44	..	..	4.90	
5 in.	..	4.62	..	..	4.83	
6 in.	..	4.57	..	..	4.57	
7 in.	..	4.61	..	..	—	

Depths.	Values of c .			
	2-in. planks.		Thin plates.	
8 in.	..	4.48 ..	..	4.48
9 in.	..	4.44 ..	..	4.08

These are the means of the values obtained for the several widths of 3 ft., 6 ft., and 10 ft., in the case of 2-in. planks, and of 3 ft. and 10 ft. in that of thin plates. It is not, as a rule, advisable to take averages upon which to base future calculations—that is to say, averages of results, although each result should be the average of a number of experimental observations of the same thing, to ensure accuracy. But in this case it may be observed, on an examination of the experiments, that there is less reason to refrain from adopting an average of lengths than of depths; the depths, therefore, and their corresponding coefficients, are given in detail, while, in respect of width of opening, the means of the values of the several coefficients are set down.

Mr. Beardmore, a very good authority, gave 5.1 for thin plates. Mr. Francis, of Lowell, U.S., gives what is equivalent to 4.81 for a plate 1 in. thick, but rounded off on both arrises, so that there is only $\frac{1}{4}$ in. flat on the top.

But for temporary stream-gauges it is hardly possible to bring the water to a still head, such as the values of c above stated demand; and probably those derived from the experiments at Chew Magna, before referred to, would give results nearer the truth in such cases. They are approximately as follow, for an iron plate 2 in. thick, flat on the top, the width of the opening being 10 ft. :—

Depths.				Values of c .
1 in. to 2 in. ..	..	..	..	.. 5.0
2 in. to 2 $\frac{1}{4}$ in. ..	..	..	..	.. 5.1
2 $\frac{1}{4}$ in. to 2 $\frac{1}{2}$ in. ..	..	..	..	.. 5.2
2 $\frac{1}{2}$ in. to 2 $\frac{3}{4}$ in. ..	..	..	..	.. 5.3
2 $\frac{3}{4}$ in. ..	..	..	..	.. 5.4
3 in. ..	..	..	..	.. 5.5
3 in. to 4 in. ..	..	..	..	.. 5.6
4 in. to 5 in. ..	..	..	..	.. 5.7
5 in. to 6 in. ..	..	..	..	.. 5.8
6 in. to 7 in. ..	..	..	..	.. 5.8
7 in. to 8 in. ..	..	..	..	.. 5.7
8 in. to 9 in. ..	..	..	..	.. 5.6

It may be observed in general that the fundamental law of water falling over a weir is the law of gravity as it affects a heavy body falling freely, which is, if we put g =the force of gravity=32, that the velocity in feet per second acquired in falling through the height h in feet is proportional to $\sqrt{2gh} = \sqrt{64h} = \sqrt{8h}$, neglecting a small decimal of .02 or .03.

The height from which the lowermost particles of a sheet of water going over a weir fall, is the height from the surface of the water above the weir, where it is comparatively still, above the crest of the weir ; and as the velocity of the particles at every point in the height decreases upwards as the square root of the depth below the surface, the mean velocity of all the particles of the sheet of water is necessarily two-thirds of that due to the lowermost, the velocity due to which is $8\sqrt{h}$, and the mean velocity of all its particles from bottom to top is $\frac{2}{3}$ of $8\sqrt{h}=5\frac{1}{3}\sqrt{h}$, by the abstract law.

But when the weir or notch through which the water runs is much less than the width of the pond above it, the sudden and various changes of direction which the particles follow offer a resistance to their free flow, so that practically not more than about five-eighths of the calculated abstract quantity passes over a weir—on an average of all circumstances—and if the different circumstances of weirs could be neglected, the quantity in cubic feet per second would be $\frac{5}{8}$ of $5\frac{1}{3}=3\frac{1}{3}\sqrt{h^3}$ per foot in length of the weir.

If the depth be measured in inches, the constant $3\frac{1}{3}$ should be divided by the square root of the cube of 12, $=41\cdot56$; but then, when depths are not great, it becomes better to take the quantity in cubic feet per minute instead of per second, and in this case $3\frac{1}{3}\times 60=200$, and $\frac{200}{41\cdot56}=4\cdot81$, the constant multiplier for these measurements. It is less than is sometimes adopted, but is as much as it is safe to adopt without taking into account all those particulars of a weir which bear upon the obstructions or facilities to the flow of water over it.

This constant 4·81 is applicable to what may be called the normal condition of a weir, in which the water falls over the weir or notch from a still head, and when the height h is the whole height from the sill of the weir to the surface of still water above it—say four or five feet above it—and when there is no velocity of approach.

When the water arrives within a few feet of the weir in a stream with some sensible velocity it must be taken into account ; it is equivalent to an increase of head forcing the water over the weir, and this head is one sixty-fourth part of the square of the velocity in feet per second with which the water flows, at its surface, towards the weir.

The condition which most influences the quantity of

water passing over a weir, when calculated from the whole height (h), is the thickness of the lip or sill over which the water flows. It is only when the thickness is reduced to a minimum, and eliminated altogether from calculation, that even an approximation to the true quantity of water flowing over a weir can be arrived at by the application of any constant, such as 4.8, or perhaps more commonly 5 or 5.1; and indeed for accuracy every weir requires a careful study of its circumstances.

The influence of thickness of lip or sill, over which the water flows, is shown clearly by some trials made by the author, the results of which were communicated to the Institution of Civil Engineers, and published in the Minutes of Proceedings, vol. xc. p. 305. One set of experiments was made over a $\frac{1}{2}$ -in. plate weir, another set over a 3-in. plank weir. The whole depth (h) varied from 1 in. to 7 in. The diminution of depth on the outer edge is produced by the retardation of the flow over the sill of the weir, the thicker sill retarding the flow of water more than the thinner one, and a thin plate, say $\frac{1}{16}$ in. thick, would, if tried in a similar way, probably show still less retardation than the $\frac{1}{2}$ in. plate. Some few experiments were made by the author with sills 6 in. and 12 in. thick, and these showed much greater reductions of depth than those with the 3-in. plank, but they were not made with sufficient accuracy to warrant their insertion along with the others in the paper communicated to the Institution. Such as they were, however, they showed a greater diminution of depth than the weirs of less thickness. The fact, however, which is the most evident upon an examination of every experiment, with whatever thickness of sill, is that there is no simple ratio between the whole depth (h) and the depth on the outer edge (d), but that $\frac{d}{h}$ varies with every

depth, in such manner that with the $\frac{1}{2}$ -in. plate $d = .72h + .025h^2$ and with the 3-in. plank $d = .33h + .05h^2$. These are in some measure confirmed by a few experiments on the outer edge of 2-in plank-weirs, made by the late Mr. T. E. Blackwell. These show that the curve which would most nearly traverse the points determined by the experiments is one of which the following would be the equation, viz., $d = .60h + .025h^2$.

The following diagram, Fig. 15, shows all three curves. The scale is one-third of the full size. By plotting the dimensions to the full size, and drawing the curve through the

points determined by the rules just given, the depth on the outer edge resulting from every whole depth may be accurately measured.

Recorders. — For the measurement of large volumes of water the notch gauge is both simple and reliable, but the arrangement as just outlined is essentially a one for temporary usage when estimating the volume of water available from any gathering-ground. Modern practice tends to the installation of a recording gear in connection with a gauge, the latter for prefer-

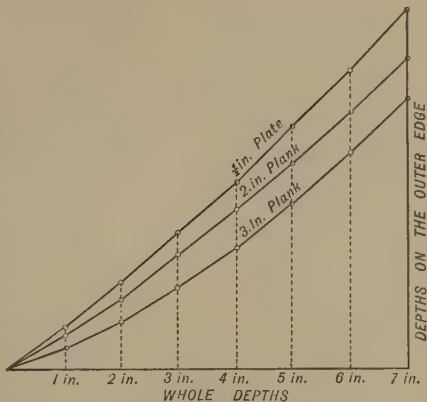


Fig. 15.

ence being formed of machined cast-iron plates set in a concrete foundation. The recorder is essentially a piece of apparatus for recording the depth of water on a time basis chart from which a very accurate estimate of the total flow over any period of time may be made. A further development is the Lea recorder, which, taking into account the formula for notch flow, actually records in cubic feet per second.

Fig. 16 shows the essential features of the Lea recorder. The rise and fall of the float *a* is communicated by rack *b* to pinion *c* on the axle of spiral drum *d*. Drum *d* is in the form of a large screw with a gradually increasing pitch. Pin *e* on the end of saddle arm *f* engages with this thread and thus imparts a lateral motion to slide bar *g* which carries pen arm *h*. At one end of the drum the pitch of the thread is zero, at the other it is maximum. This increasing pitch is made to correspond exactly with the increasing rate of flow of the water over the notch according to the formula used. Pin *e* therefore, as it moves along this groove, forms a magnified indication of the rate of flow. Scale *j* merely shows the depth of water flowing over the notch, but it is useful for checking the accuracy of the instrument at any time. It will be seen, therefore, that the machine actually does work to a formula, and the curve deduced from this formula is represented by

the spiral on the drum, which in turn controls the movement of pen *k*, which traces a curve on diagram *l*. This curve is a true and proportional record of the rate of flow.

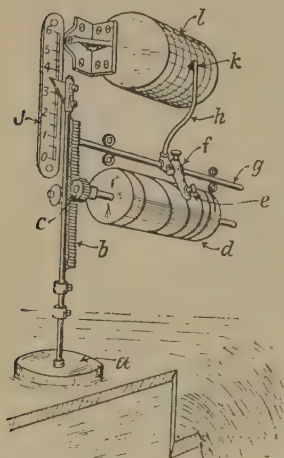


Fig. 16.

Integrating Mechanism.— The line traced on the diagram gives the rate of flow at any instant. From the diagram so traced a reliable estimate of the total flow may be made. The purpose of the integrator is to give the total flow on a set of dials. The dial box itself is much like an ordinary cyclo-meter. Attached to bar *g* it moves with it, and it thus moves in direct proportion to the rate of flow. The train wheels *a*, Fig. 17, in the counter box are actuated by toothed drum *b*, which is revolved at constant speed by a clock. It will be observed that the teeth of drum *b* are not continuous along its length,

but are cut away as shown. When there is no water flowing, dial box *c* being attached to bar *g* (Fig. 16), is in such a position relative to *b* that pinion *a* is out of mesh with the teeth on *b*. Immediately the water starts to flow, the motion of slide bar *g* brings *a* (Fig. 17) into mesh with *b*, say with

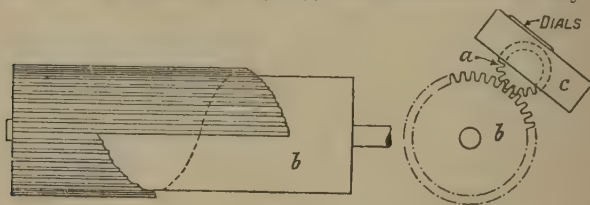


Fig. 17.

one tooth. As the flow increases to half maximum it comes into mesh with half of the teeth, and at full flow it is engaged with all the teeth, and so on according to the flow.

The mechanism, although very ingenious, is really quite simple, and although there are other recorders which effect the same end by rather different means, it is not necessary to describe them in detail.

CHAPTER VIII

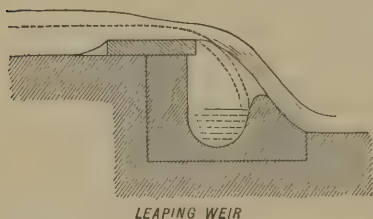
COMPENSATION TO MILLS

It is hardly possible to divert water from one district to another without injury to the interests of persons already having rights in its use, and where mills driven by water power exist below the point of diversion it is incumbent on the party proposing to divert water for use elsewhere to compensate the millowners for its abstraction. Unless this be proposed to be done in a full and adequate measure, no interference with existing rights will be sanctioned by law. Millowners are very tenacious of their water rights, and sometimes have spent enormous sums of money in opposing water-works schemes. They have often overrated the value of water power to themselves, not only in corn-mills, where water power is by prejudice presumed to be superior to steam power because of its steadiness, but in woollen and cotton mills, and even in fulling and in rolling mills, where the inequality of the work done during various parts of the day causes a waste of water on the wheels.

To give a money compensation to millowners where the number is considerable, cannot come into question for a moment. Attention must therefore be directed to provide reservoirs to regulate and economise the water, so that the quantity which previously passed the mills in floods and was useless, indeed injurious, may be stored and made use of.

Where mills are compensated from the same ground that supplies the town, it is well to have separate reservoirs for the two purposes, so that the turbid flood waters are made to overleap the channel which conveys the clear water into the reservoir for the supply of the town, and the portion due to the mills to enter that from which they are to be supplied. This may be done by making the two flood weirs at the same level, and proportioning the length of each to their respective quantities, or better by a separating or leaping weir.

Leap Weirs.—The principle on which the leaping weir acts is to separate the maximum flow of water required for the town, and to keep it at as high a level as possible, from the greater volume of flood-water, which is not required to be kept at so high a level. The smaller flow is continuous, and, up to the maximum flow required, is comparatively clear, while the flood-waters come down intermittently with a rush. The position of the lip, which separates the clear water from the flood-water, may be fixed on the following principles. For any given depth of water D in the diagram, Fig. 19, let $h = \frac{4}{9}D$, and the parabolic curve abc the line of flow of the particles which have the mean velocity of the whole sheet of water. Then, inasmuch as the height h governs this velocity, which would be, say, $7.5\sqrt{h}$ if



LEAPING WEIR

Fig. 18.

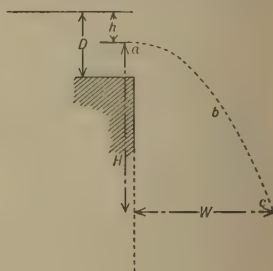


Fig. 19.

taken in feet per second, the width W , for any height H , would be $=\sqrt{4hH}$. If the water arrives at the edge of the weir with any pre-impressed velocity, the head necessary to give it the velocity must be added to that measured as D .

If d be this head, then h must be taken $=\frac{4}{9}(D+d)$. This would be the case where the water runs over a wide-crested weir before arriving at the edge over which it falls; and as this velocity of approach increases, so will the width W increase, which would then be

$$= \sqrt{4 \times \frac{4}{9}(D+d)H}$$

d being $=\frac{v^2}{64}$ when v = the velocity, in feet per second, of the water approaching the edge of the weir.

Mills generally are so constructed as to be capable of using water at the rate of not more than one-third of the average yield of a district ; whether from design or from the long experience of the millwrights of the most economical extent to which water can be applied to mills without large reservoirs, is immaterial, the fact being so ; and one-third of the available yield has generally been given to them in compensation, whereby they are abundantly benefited, for they get this third regularly day by day, whereas before they had it very irregularly.

In 1868, as well as in some other years, this allowance has had to be stopped in some cases, in order to furnish a sufficient quantity for the more urgent necessities of the towns, but money compensation has had to be substituted. A reservoir, therefore, constructed solely for the use of the mills, must have a capacity equal to one-half that of the reservoir for the town, plus about 5,000,000 cu. ft. per 1,000 acres of drainage ground, to compensate for its being deprived of the dry weather flow, which, as has been said, is received into the reservoir for the town.

Where it is desirable to take the whole of the water of a district for the supply of the town, whether because of its superior elevation or its purity, and where the mills can be compensated from a reservoir in a separate district, the area to be appropriated to the use of the mills should be of such extent that two-thirds of the yield from it shall be equal to one-third of the area appropriated to the town, because, in addition to one-third of the yield from the latter which is due to the mills, one-third of the waters from whatever area may be appropriated to the mills in the other district is already enjoyed by them ; therefore two-thirds of its yield should be equal to one-third of the yield of the district appropriated to the town ; or, in other words, the area appropriated to the mills must be one-half as large as that appropriated to the town, the amount of rainfall being the same in both districts ; and if the rainfalls be unequal, the compensation area must be more or less than one-half of that appropriated to the town accordingly as the rainfall may be less or more respectively in the one district than in the other.

It may be observed that although a given volume of water falling through a given height produces a certain number of horses-power on the basis already stated, viz. that 8·8 cu. ft. of water per second falling 1 ft. is equal to

1 horse-power—and that, likewise, the same quantity falling 10 ft. is equal to 10 horse-power—the stream of water exerts its power not for 8 hours a day only, as the horse does, but for the whole 24 hours, and is, therefore, in some cases, of the value of three times as many horses' work as its nominal power represents, if the flow of water, while the mill is standing, be stored in a dam close at hand. This, no doubt, causes a certain amount of inconvenience to mills lower down the stream, in respect of the quantity so withheld, which may not come down until a late hour in the day; and on some streams regulations are established whereby a tolerably equable flow is maintained during reasonable working hours; and taking the circumstances of a large number of mills, and comparing the work done with the power which passes them, they appear to do an amount of work, with the assistance of the night water impounded in dams, which is equal to about one-third of the power of the available quantity of water. But with a storage reservoir to control the whole available quantity and deal it out day by day during, say, 12 or 14 hours each day, the velocity of the stream is increased, and the mills get the water at earlier hours all the way down. The benefit to be derived from the construction of a storage reservoir is in respect of two-thirds of the available quantity of water. Thus, with a storage reservoir, about three times the quantity of work may be done, but the benefit derived is in respect of twice the quantity only.

The following is an instance in point. There are fifteen mills in a district above which there is a suitable site for a reservoir which commands a watershed area of 1,000 acres or thereabouts. The average annual rainfall for 14 years has been 33 in. The loss by evaporation and absorption has not been ascertained in this particular area, but comparing its position and general circumstances with other districts in which actual measurements have been made, it may be estimated with great probability at 13 in. We say we lose these 13 in. by evaporation and absorption, but it means the whole loss from whatever cause. Of course a good deal must be evaporated, and another portion absorbed by vegetation—this however being mostly evaporated from the leaves—and by sinking into the ground, from which it does not again issue within the watershed area belonging to the reservoir; but also it is very probable that over a large area the rainfall is not uniform, and that parts of it will receive more rain

than other parts, and if the rain gauge be situated in one of these wetter portions of the district, it will show a result greater than the true one. This is more especially likely to be the case where the rainfall is estimated not from a gauge planted within the district itself from which the water is to be derived, but from one at some distance from it. So that when it is said that so much is lost by evaporation, it is meant to say that the whole quantity due to the watershed area and the rainfall as shown by the gauge does not come into the reservoir. The loss varies from 10 in. to 18 in. as determined by different gaugings, and a part of this great variation may perhaps be due to the situation of the rain-gauge with reference to the actual watershed area to which its register is applied.

The question may arise, how it is known that either 10, 13, or 18 in. of rainfall are evaporated and absorbed? The result is arrived at by labour and patience in gauging the quantity of water actually flowing off the ground, by which it is found that the actual quantity falls short of the whole quantity due to the rainfall by certain quantities which vary between those limits, and comparing these various quantities with the character of the ground upon which the rain has fallen in each case, engineers exercise their judgment as to what allowance to make in any particular district. If 33 in., as in this case, be the average annual rainfall, and 13 in. the loss, there would be 20 in. available if all of it could be stored; but as that is not practicable, a deduction of about 5 in. should be made for excessive floods, leaving a practically available depth of 15 in. A depth of 15 in. over 1,000 acres would give the following quantity:—There are in 1,000 acres 43,560,000 sq. ft., which at 15 in. deep would yield 54,450,000 cu. ft. If we take 312 working days in a year, and supply the mills with water for 12 hours a day, we should be able to give out of this reservoir year by year

$$\left(\frac{54,450,000}{312 \times 12} \right) = 14,540 \text{ cu. ft. per hour, or } 242 \text{ cu. ft. per minute.}$$

But as the mills are supposed to be capable of beneficially using one-third of this already, the benefit to be derived would be in respect of 160 cu. ft. per minute only. Will it, then, pay to make this reservoir to give out this quantity for 12 hours a day all the year round, and year by year?

Seeing what many other reservoirs have cost, and not going to either extreme, this one might cost £12,000, the bank

being made as perfectly as possible, by raising it in thin layers with barrows and dobbin carts, the puddle being particularly well worked, the waste weir of good length, so that on the occurrence of a rare flood, when the reservoir might be nearly full, the overflow could not attain to such a height as to endanger the bank by over-topping it, a bye-channel being provided for the ordinary flow of the stream, and a substantial cottage built for the reservoir keeper. The falls at these 15 mills vary from 10 ft. to 30 ft., the aggregate being 326 ft. The wheels are all either overshot or breast wheels, of good construction, and in good working order. Then, although we estimated 13·2 cu. ft. of water per second per foot of fall to be, perhaps, an average of all wheels to give an effective horse-power, we shall be warranted in taking 12 cu. ft. per second in this case. This is the quantity that Sir Wm. Fairbairn frequently gave as the proper quantity in such cases. The aggregate amount of power, therefore, to accrue to the mills from the construction of this reservoir will be $\left(\frac{160 \times 326}{12 \times 60}\right) = 72$ horse-power. In the district we are considering an effective horse-power is worth £12 per annum, and if each mill contribute its quota according to the benefit it would receive (we will not now go into the more minute subdivision of benefits by considering that those mills nearest the reservoir would receive more benefit than those farther off) there will be an income of £864 a year. Deducting the wages of the reservoir keeper, and setting aside £40 a year to provide a fund for repairs, there would be a net income of £720 a year. The outlay being £12,000, the percentage return would be 6 per annum. But this does not represent the whole gain to the mill. The gain is chiefly in having a constant stream of water, and an absence of floods. Nothing is more annoying to a miller than to see water pass him which he can make no use of, and which at the same time hinders his work by backing up against the tail of his wheel. By storing the flood waters he can always reckon upon a certain power of doing work, and can enter into contracts with a certainty of being able to fulfil them. Herein lies the chief gain in constructing storage reservoirs.

CHAPTER IX

DEVELOPMENT OF WATER POWER

To a certain extent the development of small water powers is wrapped up with that of water-supply schemes because the formation of the latter may either provide suitable means for turning available power to good use, or it may, as already suggested, tend to interfere with existing rights. Another point also to be borne in mind is that in connection with rapid filtration plants a certain amount of power will be called for, and this can very often be provided by passing the water through a small turbine. Any one concerned with the construction or operation of a waterworks should always have the matter of potential power in mind, seeing that waterworks are so often far removed from sources of electrical energy.

Any running stream of water is a potential source of power, but this does not mean that all are worth or even capable of development. Water power is often spoken of as "White Coal," and the description is a very apt one.

Assuming for the moment that the cost of the necessary hydraulic machinery would compare favourably with that of any other machinery of equal power, the amount spent upon the *incidental* works of any water-power scheme, and such incidental works are usually the heavy item, represent the capitalisation of what would otherwise be fuel costs.

At the outset, therefore, in order that the development shall be a commercial proposition the interest on the capital sunk therein must be less than the average annual estimated fuel bill. In this connection it may be mentioned that these notes concern the development of quite small water powers. True the essential features and underlying principles are the same, whether it is desired to obtain 100 horse-power or 100,000, but works approaching the latter figure must be regarded rather as of national significance than in the light of mere commercial propositions. Not that national works should be undertaken regardless of their

monetary return, but rather that they can be undertaken when financed partially or wholly by the State with the knowledge that several years may elapse before the proposition pays, and whether it pays directly or not may in certain circumstances be a secondary consideration to the indirect benefit to the community at large.

Schemes Outlined.—The development of a water power has generally one of three objects in view :—

1. The supply of power to operate a near-by works, factory, or metal-refining establishment.
2. The generation of power for transmission to an industrial area or other populous district.
3. Traction.

Under the first heading the development of a few 100 horsepower may be quite an attractive proposition. It is true, of course, that such works would generally be located in an out-of-the-way place, and under such conditions it has to be considered whether the disadvantages of such a situation will not outweigh the advantages of the cheap power. Facilities for transport have a big bearing on the matter unless the raw materials were located at or near the site.

Under the second heading the development of any small scheme would have to be very carefully considered. One dominant factor is the local cost of fuel in the area to be supplied with power. In large industrial areas fuel is often plentiful and comparatively cheap, while the amount of energy consumed permits steam-operated stations to be run at the minimum cost. To bring water-generated electricity in the area means that considerable capital may have to be sunk in transmission works, in addition to the development works themselves. Thus we see no attempts to develop water power in Britain to any considerable extent. In the first place, the available powers are generally small and, after transmission to an industrial area, the cost of the energy might be little or nothing under that of steam-generated electricity. Entirely opposite conditions prevail in such countries as Switzerland, Scandinavia, Northern Italy, and South America. The available powers, while not comparable with the large American plants, are often of quite considerable size and, coal being a costly commodity, not only is the generation of hydro-electric energy a commercial proposition, but in its absence these countries would practically cease to manufacture. Therefore such developments are of national

significance, and it is such conditions which must prevail largely in the British Colonies, Ireland, Spain, and elsewhere.

Traction.—In some countries the operation of railways by water-generated electricity forms part of the general scheme of things, but to a great extent traction is a problem in itself. Beyond the electrification of local lines, which is undertaken solely with the object of speeding up transport, electric traction is practically non-existent in Britain, and at present there seems little inducement to extend it.

Abroad it is proceeding. In some countries, Spain, France, Central and Northern Europe, and South America, such developments have largely taken place through the economic pressure of the cost of coal. In America and to some extent in other locations it is largely a matter of obtaining locomotives of exceptional power. The electrification of long lengths of line is a costly business, and unless the cost of fuel for steam locomotives is abnormally heavy, and the traffic of fair volume, the commercial proposition of such ventures is always problematical even though the power itself be exceptionally cheap.

Basis of Water Power.—While the basis of steam power is so many pounds of fuel burned in the boiler per hour, the basis of water power is a product of two factors—Head and Quantity.

That is to say, to develop one horse-power a definite quantity of water must fall from one elevation to another, and in doing so pass through the turbine. Both quantity and head are to a great extent fixed by nature; an artificial head may be created and the floods of winter may be stored against the drought of summer, but the natural conditions still remain.

A small mountain stream may be the source of considerable power by reason of the head available, while a large, slow-moving volume of water may represent little potential energy.

Here enters the great dominating factor in water power, the cost of development. To proceed a step further, 1,000 horse-power is 33,000,000 foot-pounds of energy.

The head being expressed in feet and the quantity in cubic feet per second, this 1,000 horse-power may, to quote extreme cases, be obtained with a mere pipe flow of water of 8.8 cu. ft. per second at 1,000 ft. head or 1,760 cu. ft. per second at 5 ft. head, the latter flow being something in the nature of a small river.

Water at 1,000 ft. head is only met with in mountainous districts, but if this fall can be obtained with a mile or so of pipe, the turbines and pipe-line would be comparatively inexpensive for the power developed, provided that it was continuous.

The other case would involve probably an extensive barrage or a long supply canal, and the turbines to operate on such a low fall being comparatively heavy and slow-moving machines, would render the cost of such a development probably very heavy; and in view of the engineering difficulties, such as the flooding of the turbines in time of flood, such a development has every reason to prove unprofitable.

Classification of Works.—Generally speaking water power developments fall into three classes, High Head, Medium Head, and Low Head. Their limits, however, are by no means clearly defined, but good all-round values are :

High Heads—over 100 feet.

Medium Heads—25 to 100 feet.

Low Heads—5 to 25 ft.

At the present day turbines are working on heads from $2\frac{1}{2}$ ft. to 3,000 ft., but the majority probably work at heads from 50 ft. to 100 ft. At the outset the high-head plant will generally prove the best commercial proposition, no dam of any considerable size would be called for, the water would be conveyed by pipes laid over ground, and the turbines would be small high-speed machines. Such plants must, however, of necessity be located in more or less inaccessible mountainous districts, and would generally involve an extensive transmission system.

Medium-head plants are the class of development likely to find place either at a natural waterfall or at a point in a swiftly moving river where it would be feasible to erect a dam. No great length of pipe line or supply canal would be necessary; the turbines might in fact be placed at the foot of the dam or within it. For the power developed such turbines would be of reasonable size and with modern runners be capable of quite high speed, while the location of such a plant may be either near to an industrial area or itself an attractive one from an industrial point of view.

Low-head plants are only justified by special circumstances. A manufacturer would hardly be justified in

developing one, but he would be in modernising an old water-wheel site—with the incidentals existing modern turbines could be installed to give at least double the power at considerably higher speed than any old water-wheel. A large low-head development resolves itself practically into diverting a whole river through a set of large—that is large for the power developed—turbines and would generally call for an extensive barrage. Navigation or irrigation may demand such a barrage, and from the artificial head so created power could be obtained, but unless the available volume of water was considerable the power would be small and of, perhaps, little or no more than local use.

Low-head plants, again, are liable to be shut down by flood water, by reason of the working head ceasing to exist, necessitating stand-by fuel plant. The latter represents a good deal of capital.

Combined water power and fuel plants must not, however, be regarded as a bad proposition. Storage against drought may, like works to mitigate the flood evil, be of a very costly nature, the interest on whose value may greatly exceed that of the extra power rendered available. Conditions of this kind make a judiciously arranged combination of water and oil-driven plants attractive, and the practice is largely prevalent in Central Europe, while the pumping of water into a storage reservoir by the excess power at one period to be utilised during period of deficiency—the hydraulic accumulator system—is also feasible in many instances and should never be lost sight of.

The Machinery.—Under this heading it is only possible to give the briefest outline of what the modern hydraulic turbine involves. It may be said at the outset, however, that it is a highly efficient machine, a comparatively simple and rugged piece of engineering, and a very different proposition to the old water-wheel and even to what were classed as turbines twenty years ago.

There are two main types in vogue—the Pelton and the Francis. The former, technically known as an impulse turbine, is generally employed on high heads and is somewhat simpler in construction than the Francis. The latter, while of one basic design, is built in many different forms to suit a wide variety of conditions. It is often used for very high heads. The Banki turbine is a comparatively new design specially adapted to very low falls, while the Kaplan and Moody

turbines, the one German and the other American, have propeller-type runners and are specially designed to obtain relatively high speeds on low falls and so to reduce the weight and bulk of low fall turbines.

Given the necessary water at sufficient head, there appears to be no limit to the amount of power which an hydraulic turbine can be built to develop, units of 70,000 horse-power being in course of erection in America. Needless to say, however, working on heads of about 150 ft. such turbines use vast volumes of water which would only be available at large natural waterfalls.

The present backward state of water-power development in Britain, while to a certain extent the result of apathy and prejudice, is largely the result of economic conditions. With the exception of two proposed schemes in Scotland there are no extensive available powers with a reasonable operating head. Tidal schemes, and proposals like those recently made for the development of the power in the Severn Estuary, are likely to involve very costly incidental works. Their distance from industrial centres and the comparatively low cost of steam-generated power at those centres, are factors which must operate against such developments no matter how attractive they may seem upon paper. It is to be regretted, however, that more use is not made by manufacturers of the smaller available powers. Such developments should enable certain manufactures to be carried on at a sufficiently low cost to attract industries into rural districts, as such locations usually benefit the manufacturer and the community.

CHAPTER X

WATER SUPPLY FROM RIVERS—BARRAGES

IN former years the imperfect methods adopted for sewage disposal on the one hand, and the scanty knowledge available in respect of water purification on the other, rendered supplies from rivers a somewhat hazardous procedure from the public health point of view. This for the most part is now all changed. Certainly rivers in industrial areas such as the Irwell are obviously quite unfit for supplies, but it must be remembered that practically the whole of the London water comes from the Thames and Lea, and such towns as Shrewsbury and Chester draw plentiful supplies from the rivers Dee and Severn and pass it on to the consumers in a perfectly satisfactory state. To obtain a supply from a river is usually an easy matter in most instances, involving but a simple procedure of pumping to a service tank and passing the water through a suitable filtration plant. A good deal of vigilance, of course, has to be exercised to see that crude sewage does not enter the river anywhere near the intake, and sewage works effluent entering the river upstream has to be brought to a high state of purification. These remarks apply generally to cases where the volume of water at all times is much in excess of the demand. In the case of large works, especially those abroad, it may be necessary to dam a river to form a reservoir, and the work is rather different from that outlined in connection with storage reservoirs. The dam, or rather barrage, would not normally be a very lofty structure, but it may be one of considerable length and call for control apparatus of a somewhat comprehensive nature for dealing with flood water. A good instance of this is seen in the Vaal River Barrage of the Rand Water Board, and the Lake Mentz Dam on Sundays River, South Africa, the last named being described herein. It must always be borne in mind that water supply in new countries abroad often has to be under-

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taken from the broadest possible standpoint because, in addition to purely domestic supplies, such requirements as water power, irrigation, and even navigation have to be considered along with it. The dam referred to, of which a general view is given in the Frontispiece, consists of a massive ferro-concrete structure, at one end of which are five bays with substantial piers, between which are suspended five free roller steel sluice gates, each 30 ft. wide by 25 ft. deep. The piers are of reinforced concrete, and reach down to bedrock in common with the dam wall with which they are in line. A further interesting feature of this important work consists in the employment of reinforced concrete for the construction of the main columns which carry the steel superstructure and operating gear for the gates, and from which the gates themselves are suspended. In addition the counterbalance weights for the sluices are built of concrete, reinforced throughout.

The sluice gates themselves are of Glenfield & Kennedy's free roller type, the weight of each gate being about $25\frac{1}{4}$ tons, while the counterbalance weight is 53 tons, the whole being designed for a water pressure of 26 ft. head.

Each gate is built up of latticed bow-string girders spanning the opening, the latticed girders being riveted and strongly bolted to the vertical end posts of the gate. These end posts are built up of rolled steel joists, and plates, etc., and are machined parallel and on the same plane to receive and carry the rocking paths for the rollers. On the upstream side of the gates skin plating, arranged in panels, is riveted to the girders, with horizontal butt straps, the horizontal latticed girders being braced together vertically by angles and plates at intermediate points in the length, whilst the adjustable plate on the bottom edge of the gate is in lengths and machined on the contact edge. The roller paths for the gates, which are of mild steel, are also machined on all their contact surfaces, the fixed portion of the path being riveted to the gate and posts, while the rocking portions of the path are attached to the fixed path by bronze swivel pins at the bottom, top, and centre.

To allow for any possible deflection in the gates and inaccuracies in the construction of the masonry, and in the fixing of the cast-iron grooves, the rocking paths are made with a curved projection, which permits the roller path to adjust itself to the rollers by rocking with the roller motion.

The free rollers on which the gates travel when in position are of hard cast iron, and are made truly cylindrical. They are contained in a mild-steel cage, which has channel iron sides with stout fixed pins through the centre of the rollers, the holes in the rollers being bushed. The whole cage containing the rollers is compelled to travel in the cast-iron groove by means of clips which work in recesses on the sides of the path.

The cage hangs in the bight of the suspension rope, one end being attached to the gate and the other end to the top of the cast-iron groove. Its travel, therefore, corresponds to half the travel of the gate. The staunching bars for the gate side joints are of the solid drawn type. They are machined on the external surface, and are loosely suspended from the gate by chains, so that they are sensitive to water pressure when the gate is closed, and bear on the machined faces which are attached to the gate and the groove respectively. The grooves themselves are of cast iron. They extend from the floor to the top of the masonry piers, and are erected in lengths and machined where any two surfaces come together, all the joints being provided with fitted bolts, and the lower portions of the side groove being fitted with gunmetal faces for the staunching bars.

The pulleys for the counter-weight ropes are of cast iron, machined for the rope, and are furnished with long roller bearings. The guides for the counterbalance weights consist of channels, which are attached by binding bolts to the superstructure. The two hoisting ropes for the gates are of galvanised steel, and have a net breaking stress of about 66 tons each. The suspending shackles are adjustable, so that they can be set towards the back or the front to ensure the gate hanging vertical. The ropes have spliced-in solid eyes at each end, and are provided with the necessary adjusting screws, which are of sufficient length to take up the "stretch" and to lower the balance weight on to the piers when it is desired to remove a gate from its groove for repairs.

The gearing is of the rapid-lifting conical-drum type, the winding drums being of cast iron, with helical grooves machined in them for housing the hoisting ropes. Its main bearings are equipped with self-aligning roller bearings specially designed so as to permit of as much freedom as possible in working, and each runs in an oil bath carried in a dust-proof case. The gearing has both spur and worm reduction arranged in

a dust-proof headstock and running in oil. It is operated by double crank handles. To permit of the gates being rapidly raised to cope with an approaching flood, each gate is provided with a trip clutch, which, when released by hand, permits the heavy counterbalance weight, which, as will have been noticed, is just over twice the weight of the gate, to descend, thus automatically raising the gate. The speed of falling of the gate is governed by a special time control gear, which works in conjunction with the helical grooves on the drums, the latter being so arranged that the diameter at the edges is greater than at centre of drum, so that in consequence the gate is automatically retarded as it approaches the end of its travel, and is finally brought to rest when the limit of travel is reached. By this arrangement, any shock or jar on the completion of the travel is, it is claimed, entirely eliminated.

The whole arrangement is designed to give two lifting speeds :—

(1) Rapid automatic, both when lifting and closing, which is obtained by withdrawing the free roller clutch by means of the handle on the headstock.

(2) Slow speed by hand through the ordinary crank handles.

The gates can be locked in any position of their travel—when the clutch is engaged—by means of a pawl.

To facilitate the erection of these large gates at the site, there was a specially designed travelling gantry, arranged to run on a rail track laid alongside the barrage on the pier tops, and having the form of a carriage to support each gate during transit. In addition to the longitudinal travelling motion, small carriages are arranged to travel on the main structure of the crane in a transverse direction, so that each gate can be loaded or unloaded from the crane at any desired bay in the barrage, or hauled to the bank as desired.

Among the features of the complete design, to which the makers draw special attention, is, in the first place, that the gates and all the accessories are constructed throughout to give a maximum degree of water tightness with a minimum of friction in working. The free roller design of gate is, it is pointed out, absolutely essential for conditions such as are met in the case of the Sundays River dam, and it is characterised by the fact that, when the sluice gate is raised clear of the water, the staunching tubes and rocking paths are also lifted clear with it, and can be readily examined or repaired.

Furthermore, the roller trains are so constructed that repair is a comparatively simple matter. Great attention has, because of the exposed nature of the work, also been paid to the question of lubrication. All the bearings intended to relieve friction are provided with rollers or balls, and all the gears are continuously submerged in oil, while all working bearings are housed in dust-proof cases, the result being, so it is stated, that the weight immediately responds to manual power, even when applied after very long intervals. The "quick lifting" arrangement is described as being particularly effective, as it enables the attendant rapidly to open one or all of the sluices in event of heavy floods coming down the river, while the degree of opening of each sluice can be perfectly controlled at the operating geared headstocks, as, for example, when scouring out.

These works, and more particularly the sluices, have been described at some length, because they constitute the modern conception of a barrage. There are but few works of this class in Britain, and these are mainly connected with river regulation. Conditions abroad, however, are often very different because, as previously stated, the "Water Question," as Mr. Slagg aptly terms it, has in new countries to be regarded from the very broadest standpoint. With large unpolluted rivers providing enormous volumes of water a river barrage as a source of supply is often an attractive alternative proposition to a gathering-ground on account of the potential power available. The power may not only be of almost as great a benefit to the community as the water supply itself, but it may be judiciously utilised in part to pump water to outlying districts. The whole scheme may under such circumstances be a prolific source of revenue.

CHAPTER XI

WATER SUPPLY FROM WELLS

THE subject of wells is a specialised one. In the first place, it is rather difficult to treat on paper, and secondly, it is one which is dealt with by many able writers who are specialists in this field, so the following remarks are only intended to be brief and of general application. A considerable amount of water is obtained, especially in the eastern and southern portions of this country, from wells, and in the Midlands the South Staffordshire Waterworks Co. is practically wholly concerned with the operation of a large number of pumping stations in connection with wells and boreholes. While often disliked by steam users there is a good deal to be said in favour of well water. It usually eliminates any form of treatment, and the sinking of a well, the erection of the pumps, and the relatively short lengths of main required, often render it a cheaper means of installing a supply than a surface-water system would be ; but against this has to be put the continual cost of pumping and the possibility of the underground supply being ultimately "worked out." Amongst large private consumers wells are quite popular, and the water is often suited to certain trades purposes. It is a popular fallacy that the excellence of Dublin stout is due to the use of Liffey water, whereas, as a matter of fact, the breweries have their own wells.

On the other hand, the small private well is often nothing more than a stinking abomination. Lined with defective brickwork and placed close to houses it is liable to infiltration from the cesspool, if there is one, and almost certainly from adjacent manure heaps which figure on any agricultural holding. The water pumped from it would not be tolerated for one moment by any officer concerned with the public health, and the reason why the users are notoriously free from water-borne diseases is because their systems have become hardened to contami-

nated water. The lowering of general health, however, goes on unsuspected, and these notes are for the guidance of those who seek to improve rural conditions by the provision of at least drinkable water. It cannot be too strongly urged that, if at all possible, a supply should be laid on from a public main, the cost of even a considerable length of pipe being well worth while. If, however, a well must be resorted to it should be properly constructed.

Preliminary Investigations.—It is a difficult matter to briefly treat on paper the local conditions which are favourable to the sinking of a well or the driving of a borehole.

There is, of course, the water diviner, variously regarded as a scientist or a witch; his services, however, are undoubtedly of value, and unless local conditions bespeak a distinct probability of water being obtained at no great depth, a water diviner's advice should be sought. A borehole is rather another matter. A well in this connection may be regarded as an excavation up to, say, 50 ft., cut by hand and lined in approved fashion. Local well sinkers can be usually found to undertake the job. A borehole is relatively small in diameter and is cut by special drills, etc., and is work which can only be satisfactorily undertaken by firms who have the requisite equipment. The principle, however, of all wells is the same, that is to say, they are cut to reach subterranean water within the earth's crust; this subterranean water exists, but whether at 10 ft. or 1,000 ft. below surface is a matter of local circumstances. The matter is made clear by Fig. 20. This shows a section of an area of land in which porous and impervious strata are supposed to exist. Rain, which is the basis of all fresh water, falling on the surface naturally sinks through the porous strata, and that portion not absorbed by plant life, or draining into the nearest watercourse, percolates till it is held up by the layer of impervious strata and collects where the shallow well will tap it.

This generally is the condition under which local wells are sunk, and it has the advantage that water is obtainable at no great depth. The trouble, however, is that the supply may fail in summer, and the sinking of another well in the vicinity may curtail the supply. It is this class of well which is liable to pollution, and this is quite easy to understand when it is remembered that the seepage from a cesspool or the drainage from a manure heap is bound to find its way into the

well. There are two very necessary precautions, therefore—the first to sink the well as far as practicable from such sources of pollution, and to see to the proper disposal of the refuse. The old leaking cesspool is in any case out of date. The modern septic tank, with its watertight walls and bottom and the overflow led away through a sub-irrigation drain, effects a certain amount of purification, and with a reasonable distance between the irrigation drain or filter, if this is impracticable, the chance of pollution is small; but a yearly examination of the water by an analyst is a wise precaution costing but little. It might be argued that the water could be purified, but in the writer's experience small filters very

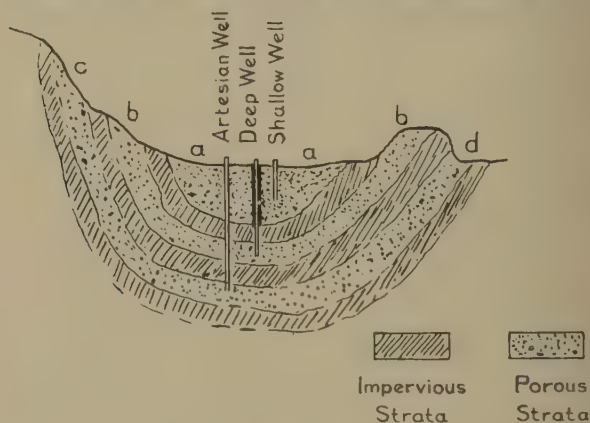


Fig. 20.

soon cease to function as filters, merely becoming breeding-grounds for bacteria, and are well left alone. If the only available water is pronounced dangerous, then there is no option but to boil all that is used for potable purposes, chlorination, like filtering, being impracticable on a small scale.

Deep Wells.—The water supplied by a deep well, as shown on the diagram, is obtained under entirely different conditions. Commencing as rain-water falling at *bb*, it may travel a considerable distance before arriving at the well bottom. This alone is sufficient to ensure its purity, while the impervious strata through which the well is driven protects the water from local pollution provided that the heavily lined portion

is formed of brick in cement, concrete, or cast-iron tubing. The water will probably be excellent for drinking purposes although it may be hard. It is usual, however, to make provision for the storage of rain-water, and here again money spent on a good big tank is money well spent. Not only is the well-water conserved during dry seasons, but the saving in soap and the incrustation of boilers, etc., is alone worth the cost.

The rain-water falling at *c* gives rise to yet a third condition. Collecting at the base of the lower porous layer it would cause the spring at point *d*, while the borehole will tap the subterranean water. In this connection, owing to the higher level of the land at *c*, the water may rise to the surface and may, under very favourable conditions, issue thereat under considerable pressure, forming an *artesian* well.

Some Notes on Construction.—It is not generally advisable to entrust a well-sinking job to a handyman or any local jobber, and the following notes are intended to be for general information only. While the work sounds simple enough, that is excavating, placing the curb, building up the brickwork, and sinking it, it is essentially a tricky business, and the well sinker is a craftsman who alone understands it. Generally, after excavation has been carried a reasonable depth, an iron curb is let down into the excavation, levelled up, and part of the brickwork built up on it. Further excavation below the curb causes it to sink, and so the brickwork is built up course by course. In place of brickwork cast-iron segmental sections may be used, and although rather costly, they are much to be recommended. A borehole, the average diameter of which is about 9 in. to 20 in., is cut by a succession of blows with a form of chisel. Into the hole a length of pipe is driven, the dirt removed by another special appliance, another length of tube added, and so the sinking proceeds. The tools are operated by an arrangement something like a pile driver. Other methods again employ rotary drills, which cut out a circular core and draw up from a bore through rock, cores several feet in length. Pumping from a bore, if it does not function as an artesian well, is a special problem, and the matter of pumping from wells will be dealt with in a subsequent chapter. In the meantime attention is directed to the possible supply at point *d* from

Springs.—The value of a spring is not always fully appreciated. A crude collecting trough often forms the sole means

of putting it to use. Little or no provision is made to ensure that the supply is kept clean, and the winter surplus is allowed to run to waste without thought to the spring drying up in midsummer. A spring is generally found located on a hillside and surrounded by swampy ground. The water is generally of good quality, and the soft earth should be excavated back into the hill to reach the impervious stratum, while the yield may often be increased by trenching round about, laying tile drains in the trenches leading to the collecting chamber, and filling in over them. A proper collecting chamber is really essential. It may be quite cheaply constructed in concrete, and it serves to store the water and also to effect a certain amount of purification in the shape of allowing suspended matter to settle. In the case of small supplies, the location of a spring may permit of some of the consumers being supplied by gravity, those at a higher level being connected to an elevated tank from which the necessary water would be pumped. The foregoing notes touch but briefly upon the subject of wells, and deal in the main with small ones. In connection therewith it is most necessary to direct attention to the very dangerous state of affairs which still exist in many rural districts. People who can draw water from a well are nearly always averse to bearing the cost of a proper supply, but it is the duty of a water authority, in the interests of public health, to bring pressure to bear to enforce this.

CHAPTER XII

CONDUITS

WATER may be very well conveyed along a hill-side in an earthenware pipe, in moderate quantities. A 24-in. pipe running half full, with an inclination of 5 ft. in a mile, will carry a million and a half gallons a day, and $2\frac{1}{2}$ millions when running two-thirds full. But to go to a smaller size, a 12-in. pipe with the same inclination would carry 300,000 gallons a day, half full, and 450,000 gallons if running two-thirds full. A 27-in. pipe will carry, at the two degrees of fullness mentioned, 2,300,000 gallons, and 3,400,000 gallons, respectively. A 30-in. pipe, nearly 3 million gallons a day half full, and $4\frac{1}{2}$ millions if running two-thirds full.

Pipe Flow.—The carrying capacity of any circular pipe may be found for any degree of fullness from the following considerations. If full, or half full, the hydraulic mean depth is half the radius. This is evident, because the cross-sectional area of the pipe is equal to that of a triangle, the base of which is equal in length to the circumference of the circle, and the height to the radius, half of which, multiplied into its base, is the area. The hydraulic mean depth is the area divided by the whole circumference when the pipe is running full of water, and is, therefore, half the radius; and it is the same when the pipe is running half full, for in that case it is half the area divided by half the circumference; but at any height, H , above or below the centre of the pipe, it is as follows— D representing the diameter, and R the radius:—The width at the surface of the water (see Fig. 21) is $W = \sqrt{R^2 - H^2} \times 2$.

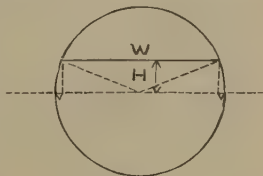


Fig. 21.

The width V is $\frac{D-W}{2}$. From H and V find the length

of that part of the circumference in contact with the water above the centre of the pipe, and add it to the lower half of the circumference, thus procuring the wetted border of the section. The length of the two arcs above the centre is the same as that of a single arc the chord of which is $2H$. Find

the chord of half the arc $=\sqrt{H^2+V^2}=C$. Then $\frac{8C-2H}{3} =$

the length of the two arcs above the centre, to be added to that of the lower half of the pipe, to find the wetted border, which, being multiplied into half the radius, gives an area, to which

is to be added $W \times \frac{H}{2}$ for the whole sectional area of the stream, and this divided by the wetted border gives its hydraulic mean depth.

For example, in a pipe 24 in. diameter running two-thirds full, the height H is 4 in., and the width $W=\sqrt{12^2-4^2} \times 2=22.62$.

The width V is $\frac{24-22.62}{2}=.69$.

The chord of an arc above the centre is $C=\sqrt{4^2+.69^2}=4.05$.

The length of the two arcs is $\frac{8 \times 4.05 - 2 \times 4}{3}=8.13$.

The lower half of the circumference of the pipe is $\frac{3.1416 \times 24}{2}=37.70$, and $37.70+8.13=45.83$ in. $=3.82$ ft., the wetted border.

The sectional area is $45.83 \times 6 + \frac{22.62 \times 4}{2}=320$ sq. in. $=2.22$ sq. ft., and the hydraulic depth, or mean radius, is $\frac{2.22}{3.82}=.58$ ft.

If Eytelwein's rule be adopted, in which h represents the hydraulic mean depth, and f twice the fall per mile, the mean velocity of the stream of water would be $\frac{10}{11} \sqrt{hf} = \frac{10}{11} \sqrt{.58 \times 10} = 2.18$ ft. per second, or 130 ft. per minute, and the area being 2.22 sq. ft., the quantity is 288 cu. ft. per minute, or 2,592,000 gallons per day as before stated.

Larger pipes than those of about 24 in. diameter are difficult to handle, require heavy tackle to lift about, and are

liable to split longitudinally with external pressure, unless the pipes are evenly bedded all round the lower half, and the haunches of the top solidly filled in between the pipe and the sides of the trench.

At what diameter, exactly, a brick or stone conduit becomes cheaper than a pipe, depends upon the local materials ; but, usually, about 30 in. becomes the turning-point, in respect of expense, between an earthenware pipe and a culvert of masonry. The pipe has some advantage in being glazed, but for clear water this is not of much importance. A circular conduit, 2 ft. diameter and half a brick thick, would usually be laid at less expense than a pipe of the same size. Very excellent conduits are made of square shape, with flag bottom and covers, and sides of clean bedded stones with squared joints. A larger quantity of materials is required for this form than for a circular one, because to carry the same volume of water at the same inclination, a larger sectional area is required, and the walls, being straight, need to be thicker.

Channel Flow.—The question arises, when considering the advisability of adopting a square or a circular form, what are the comparative carrying capacities of the two forms, the more immediate question being what dimensions of the square form carry the same quantity of water per day or per minute, as a circular conduit of given diameter. In rectangular channels the best proportion of width to depth of the stream is width 2, depth 1. That proportion is the best which gives the greatest hydraulic mean depth with a given quantity of materials used in the construction of the channel, because, the greater that depth, the less need be the sectional area of the stream, the carrying capacity of all conduits having the same inclination being as the area multiplied into the square root of the hydraulic mean depth. A rectangular conduit may be compared with a circular one in the following manner :—Let them both run half full ; then if the height of the rectangular conduit be the same as its width, the width of the stream will be twice its depth. Let h represent the hydraulic mean depth, as before, W the width, and D the depth of the stream (see Fig. 22). If the width be twice the depth, the area is $2D^2$, and $h =$

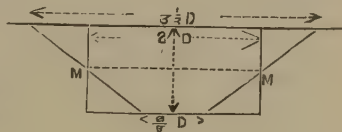


Fig. 22.

$\frac{2D^2}{W+2D} = \frac{2D^2}{4D} = \frac{1}{2}D$; that is, in a rectangular stream in which the width is twice the depth, the hydraulic mean depth is half the simple depth, as, in the circular form, it is half the radius. Then $\sqrt{h} = \sqrt{\frac{1}{2}D} = \frac{1}{\sqrt{2}}\sqrt{D}$.

The sectional area multiplied into the square root of the hydraulic mean depth thus becomes $2D^2 \times \frac{1}{\sqrt{2}}\sqrt{D} = 1.41\sqrt{D^5}$, and this quantity must be the same in the square as in the circular form, to satisfy the conditions.

For example, in the case of a rectangular stream, in which the width is twice the depth, what would be the dimensions to make the carrying capacity equal to that of a circular conduit 3 ft. diameter running half full? In this latter form the hydraulic mean depth or mean radius is .75 ft. The area

of the lower half of the conduit is $\frac{3.1416 \times 3 \times .75}{2} = 3.53$ sq. ft.,

and the area multiplied into the square root of the hydraulic mean depth is $A \times \sqrt{h} = 3.53 \times .866 = 3.05$. Then in the

rectangular channel $A \times \sqrt{h} = 1.41\sqrt{D^5} = 3.05$. $D^5 = \left(\frac{3.05}{1.41}\right)^2$

$= 4.66$, and $D = \sqrt[5]{4.66} = 1.36$ ft., the depth, and the width is 2.72 ft., the area being 3.70 sq. ft. The border is $W+2D$

$= 5.44$ ft., and the hydraulic mean depth is $\frac{3.70}{5.44} = .68$ ft., being

half the depth, as before stated. If the quantities of water carried by the two conduits be calculated from these data, they should be the same.

Small conduits are more liable to freeze than large ones, and should be covered, to protect them from freezing as well as from dirt. Snow is troublesome in an open conduit whether large or small; it clogs the run of water; but the freezing over of the surface of a large body of water is not of so much importance as in a small one. In either case it reduces the sectional area of the stream, and at the same time increases the solid surface with which the water runs in contact, but the effect of this in reducing the quantity of water carried is but little in a large conduit, while it is very considerable in a small one.

Construction.—In respect of protection from dirt, fencing may be sufficient in the open country, but the expense of fencing bears a greater proportion to the whole expense in a small than in a large conduit, and is almost as much as that

of covering a small conduit. If the line of conduit is not subject to contamination by dirt it may be open, though it be not large, if the inclination is sufficient to cause a velocity of about 3 ft. per second. This would wash away earth if not protected by a facing.

In an open conduit cut with side slopes at which the ground will stand without slipping, so that the fencing is for the purpose of protecting the earth from the wash of water rather than for keeping it up by its weight or lateral resistance, the side slopes would usually be flatter than those which coincide with the best form in respect of carrying capacity. They would usually be $1\frac{1}{2}$ to 1 or more, but side slopes of $1\frac{1}{3}$ to 1 are best in respect of carrying capacity, which, with these slopes, is the same as that of a rectangular channel of a width equal to twice its depth, and of the same sectional area. The peculiarity of this slope is that the length of the two slopes and bottom is the same as the length of the two sides and bottom of a rectangular channel of the proportions mentioned, the area of the two forms being the same; consequently the hydraulic mean depth is the same, and, therefore, the discharging capacity. If the slope be drawn through the point M in Fig. 22, it will leave the area of the section the same as that of the rectangular channel, whatever the slope be, but there is only one slope which will leave the length of border the same as that of the rectangular channel; that slope is $1\frac{1}{3}$ to 1, and the length of the two slopes is equal to the top width, being $3\frac{1}{3}$ times the depth, the bottom width being $\frac{2}{3}$ of the depth. Referring to the figure, the area is $\frac{3\frac{1}{3}D + \frac{2}{3}D}{2} \times D = 2D^2$, and the border is $1\frac{2}{3}D + \frac{2}{3}D + 1\frac{2}{3}D = 4D$.

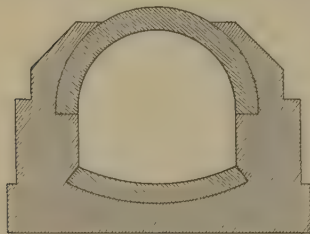
Hence, the hydraulic mean depth is $\frac{2D^2}{4D} = .5D$, the same as that of the rectangular channel, and also of the circular one.

But sometimes practical considerations have greater weight than the best deductions, and the sides of an open channel would generally be less troublesome to keep up with slopes of $1\frac{1}{2}$ to 1 or more. Frost is the great disturber of the upper part of open channels, whether large or small, although the covering of water prevents it from reaching the bottom. The sides above the water should be porous—the facing, that is to say, should not be close-jointed—so that water may not accumulate behind it, for when it does so, it swells when frozen and disturbs the facing. But when the facing is thus

open-jointed it would be the means of introducing mud into the conduit from behind it in long-continued rains upon clayey ground; to prevent this, the facing should be laid upon a bed of sand.

Through loose ground the making of the conduit watertight must always be one of the chief difficulties—a difficulty of course to be overcome. For this purpose puddle, to be used with all materials except concrete, is perhaps the least expensive material, and gravel puddle is better for the purpose than clay puddle. A foot is an abundant thickness for small conduits for the bottom and slopes of the ground, and 2 ft. above ground in the embanked portions of the line of conduit. In these the hardest of the excavated earth will be placed along the central portion, and brought up to the level of the under-side of the puddling, as far as the height above ground is but a few feet.

A material which may be used for conduits is concrete, made with Portland cement of good quality. It forms at once a watertight channel and one which needs but little protection of the surface, a facing of cement being sufficient for all that part under water, but above the water, in an open conduit, it would be preferable to face the concrete with brick or stone, and indeed for some little depth below the water surface. A covered conduit may equally well be made with concrete, both bottom



COVERED CONDUIT

Fig. 23.

and top, and the thickness need not be more than about half as much more as would be the proper thickness of brickwork.

Crossings.—Beyond the several points in a long line of conduit at which slight cutting should end, or the height above ground would be but a few feet, and where it would be economical to cross straight over low ground rather than extend the line far up the hollow and back again on the other side, the aqueduct is best carried on piers and arches; but economy points to girders, which can be made to carry the water between them on a floor laid upon the lower flanges, or in the form of a wide box-girder; but if the water runs in contact with the iron the conducting property of the metal quickly draws heat from the water in winter. If it be lined with a non-conducting

material, which is at the same time brittle, the difference in the rate of expansion by heat of the metal and the lining tends to destroy the adhesion. This may in some measure be prevented, and perhaps entirely, by sinking the aqueduct bodily as far as is necessary to completely immerse it, so that it may preserve a nearly uniform temperature both top and bottom ; but this, of course, somewhat adds to the retardation of the flow by increasing the wetted border. To prevent contact with the metal altogether, by substituting an independent carrier for the water within and clear of the girders, would be expensive, but such a carrier might very well be made in earthenware, either rectangular or circular, and of any size by being jointed crosswise.

The least expensive aqueduct would be an earthenware pipe—of stoneware or fireclay—carried on a single girder in saddles upon the top flange. An open girder, continuous over the piers, with wide flanges both top and bottom, would resist the wind, and as the pipe and the water together would be of considerable weight, it would not be likely to be blown off the girder, especially if tied down with a strap over all, every 10 ft. or so.

However desirable it may be to follow the contour of the ground, there will occur, in any conduit of considerable length, breaks in the continuity of such a line, caused by an intervening valley, or by a range of hills, through which a tunnel may be necessary, in order to continue the conduit. In the case of a valley to be crossed, cast-iron pipes are used, laid underground at such a depth that they may have a sufficient covering for their protection (say 2 ft.), and rising again on the opposite side to within so much of the level of the conduit from which they depart as will give a sufficient head to the water to force its way through them. The following formulæ by which the necessary difference of level of the two ends of the pipe, or the head, is ascertained, are based on the experiments of Dubuat, Bossut and Couplet. Taking a number of their experiments made with pipes of from 1 in. to 5 in. in diameter, and from 30 to 1,700 ft. in length, M. Prony found that, taking English measures and putting

h = the head of water in feet,

l = the length of the pipe in feet,

d = the diameter of the pipe in feet,

v = the velocity in feet per second $= 48.49 \sqrt{\frac{dh}{l}}$ and by

substituting a fall in feet per mile, which call f , $v = \sqrt{\frac{df}{2.24}}$, and $f = \frac{2.24v^2}{d}$.

Eytelwein, taking the same set of experiments, found that $v = 50\sqrt{\frac{dh}{l+50d}}$. This correction $50d$ is for short pipes, but in case of actual waterworks practice it may be neglected, and then Eytelwein's expression may be translated into $v = \sqrt{\frac{df}{2.11}}$ and $f = \frac{2.11v^2}{d}$.

In using these and similar rules for practical purposes, engineers have, in exercising a wise precaution in under-estimating rather than over-estimating the capacity of a pipe to deliver water, had a tendency to increase these coefficients of 2.24 or 2.11, for the purpose of allowing for the obstruction of bends and other irregularities in a long length of pipe, and Mr. Blackwell proposed 2.3 in all cases; but a few of the more eminent water engineers, who have had large experience in gravitation works, have been able to ascertain from actual examples the quantity of water passing through long and large conduit pipes, and Mr. Bateman has said that the very general method of coating pipes inside and out with the pitch of coal-tar, to preserve them from oxidation, has had the effect of diminishing the friction of the water to a considerable degree; so that it is probable that when pipes are so coated the effect of it is to facilitate the passage of the water to quite as great an extent as the bends and other irregularities retard it. Roundly, 10 ft. per mile may be named as the allowance for the difference of level of the two ends of a pipe thus laid across a valley; but where the pipe is large, less than that is sufficient. Ten feet per mile will induce a velocity of 3 ft. per second through a 2-ft. pipe.

It frequently happens that in a length of several miles of conduit pipe one or more streams of water, or ravines, or other such places, have to be crossed, and instead of passing under them, the pipes are sometimes carried over them, as shown in the illustration (Fig. 24).

Large pipes are now commonly cast in 12-ft. lengths, so that a stretch of 36 ft. can be had from side to side, and if a greater length be required, one or more intermediate piers can be built. It is not usual to carry more than three pipes in one stretch; but three are safely thus carried across such

places by means of flange joints strongly bolted, the flanges themselves being also of more than the ordinary strength.

As to the strength of cast-iron pipes laid across a valley, it is necessary to consider that concussions may occur if stop valves be placed on the line, and also that concussions may occur if the pipe be so laid as to be liable to air locks, although no concussion would occur from either cause, if in the one the valves were made with a pitch of screw so small as to prevent the valve door being shut down suddenly, and, in the other case, if one or more self-acting air valves were placed on the summit of every rise. The most trying time to a pipe of this kind is when it is first charged with water; but it is easy to admit the water for the first time gradually, and not recklessly to open the valve at the head of the pipe, as if it were already full of water. When these precautions are taken, there is no need to make the strength of a pipe

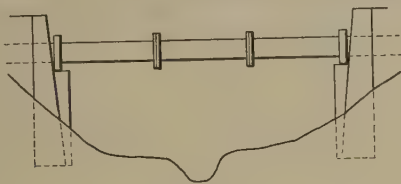


Fig. 24.

equal to more than six times the strain it is subject to from the head of water upon it; and if we take the low estimate of six tons per square inch as the utmost tensile strength of the iron, we shall have a working strain of one ton per square inch of metal.

At the head of a line of conduit pipe there should be a sluice valve, and a provision made for turning the water sideways into some proper watercourse. The mouth of the pipe should be some feet below the head of water—10 ft. is not too much—to prevent air being drawn into the pipe during the ordinary working. When this valve is shut down, the body of water shut into the pipe will proceed on its course at first with the momentum it had acquired, leaving a vacuum between its head and the sluice valve, which will cause it to partially return, and to oscillate backwards and forwards until the forces within the pipe are balanced.

At every low point a washing-out pipe should be placed, with a valve upon it, so that the conduit pipe may be emptied

when necessary. It is well to have these outlets of limited size, so that carelessness in opening them suddenly may not cause damage to property on the brook course below, if it be a mere brook into which the water is turned.

It will not unfrequently happen that in the course of a long conduit pipe a railway will have to be crossed, and here the pipes should be unusually thick—capable, say, of withstanding ten times the strain they will be subject to.

In cutting the trench for the pipes, air locks should be as much as possible guarded against by cutting through many small elevations of ground, so that long reaches of rise and fall may be obtained. The desirability or otherwise of this certainly depends on the perfection of action of the air valves; for if this action be perfect the more ups and downs the better, for the air has then shorter distances to travel before it can escape; but, considering that a large quantity of air is contained in the water itself which is constantly seeking a higher level, it is perhaps better to afford it as few points of accumulation as possible. The best known air valves are those with a ball of gutta percha falling down from its seat as long as it has nothing but air to support it, the air meanwhile escaping, when pressed by water, but rises to its seat by floating on the water when it reaches it, and as long as no fresh supply of air accumulates about it it is held up tightly to the india-rubber seat provided for it. These valves are made by Messrs. Guest and Chrimess, of Rotherham.

In a discussion on a paper on the Melbourne Waterworks, Mr. Hawksley stated that concussions in mains are caused by the sudden escape of air at the air valves, causing two columns of water to meet each other and give a blow to the pipe, and in one case he ordered the air outlet to be no more than $\frac{3}{4}$ in. diameter, so that the air could escape only gradually.

Concrete Conduits.—Present day methods tend to the adoption of ferro-concrete for the construction of large conduits. For those which do not operate under pressure that class of pipe with slip joints, now made in large quantities for sewerage work, is entirely suitable, being obtainable when required in very large sizes. For pressure pipes other methods have to be resorted to. The first of these is the construction of the conduit by the continuous method. This is ferro-concrete work pure and simple, and involves the setting up of the necessary forms and centering, placing the reinforcement, usually in the shape of longitudinals and hoops, as

demanded by the conditions of pressure, and placing the concrete by accepted methods. It is quite a satisfactory method of forming a conduit, but recent devices for jointing lengths of reinforced concrete pipe to withstand pressures up to 200 ft. or so have brought into favour sectional pipes for this purpose. One system makes use of a thin metal cylinder as part of the reinforcement. The pipe lengths are cast in special moulds, sometimes centrifugally, with the aforementioned tube projecting. With the pipe in place a metal collar is placed over this projecting tube and leaded up. Into it on the other side the projecting portion of the next pipe is inserted and also leaded up, the whole being subsequently covered in concrete. Such is the basic principle of the Bonna pipe. In America the lock-joint system is favoured. This involves the casting of the pipe lengths with cast-iron ends, to which the reinforcement is tied. These iron ends are machined to a special form of spigot and socket. Into the latter is placed a ring of lead pipe filled with gasket and sweated up as a continuous ring. When the spigot end of the next pipe is inserted and *drawn* in, the lead pipe is compressed into a wedge-shape groove in such a way that the joint is watertight and remains so, while at the same time capable of adjusting itself to expansion, contraction, and settlement. The subject is dealt with at some length in the writer's *Development of Water Power*.

CHAPTER XIII

SERVICE RESERVOIRS

THE greatest difficulty usually met with in the construction of service reservoirs is that of site and elevation. The remark made by somebody that it was curious how rivers mostly ran through towns must have had its origin in a forgetfulness of the circumstances under which towns have grown up. Towns have begun to be formed on the banks of rivers more or less navigable, but it was the navigability of the river that first induced settlement on its banks for the purpose of commerce; and then, up to the time that steam became known to be so powerful an agent, and as roads began to be made, waterfalls were sought for power for manufactures, as well as for manufacturing uses. So that we find the "centres" of all manufacturing towns close to the rivers running through them; but presently people build their houses outside and away from their factories and warehouses, and therefore on higher ground, and it is this tendency to get to higher ground on which to build houses, that renders the choice of a sufficiently elevated site for a service reservoir difficult to find within a moderate distance of the town. The service reservoirs of Manchester are five miles off; those of Liverpool eight miles; and those of the most feasible scheme once proposed for the supply of water to the metropolis were to be ten miles off. However, those of most other towns are nearer. When the service reservoir is made near the town it is recommended that it be covered, to protect the water from the impurities of the atmosphere; and in all cases a service reservoir should be covered if it is less than 15 ft. deep, for if still water be exposed at a less depth than that, vegetation is promoted, and when that dies animalculæ are formed.

The size of a service reservoir, unlike that of an impounding or storage reservoir, depends upon circumstances within the immediate control of the engineer—that is to say, it depends upon the character of the supply. If the supply be through

a conduit from a storage reservoir, and the conduit be a short one, one or two days' supply may be sufficient, because anything happening on such a conduit to prevent the delivery of water can soon be put right; while the longer the conduit, or the more complex in character, the greater should be the contents of the reservoir. The service reservoir of the scheme already mentioned as designed for the supply of the metropolis, the conduit of which was to be 180 miles in length, was intended to hold three weeks' supply. Also where a service reservoir is supplied by pumping engines, it should hold a considerable quantity of water, to allow of stoppages of the machinery or part of it. Generally, perhaps from three to seven days' supply may be said to be the ordinary practice.

Reservoirs not requiring to be covered are usually formed by excavating the ground and embanking the earth round it, adjusting the level of the bottom of the excavation so that the quantities of cutting and embankment are equalised. To render the reservoir watertight, it is usual to lay down a flooring of clay puddle, 18 in. to 2 ft. in thickness, worked in layers in the manner described for the puddle of the storage reservoir. The slopes of the excavation should be not less than 2 horizontal to 1 vertical, and should be cut down in steps, so as to hold up the puddle placed on them. From the top of the slope of the excavation the puddle is continued on the surface of the ground (the bed-puddle) to the centre of the bank, and up through the centre of the bank the puddle is carried as a wall. In reservoirs of considerable size it is necessary to lay down this bed-puddle, or over a part of its area, before the floor-puddle and that on the slopes can be completed, in order to provide a place of deposit for the excavated material, but a margin of it, of not less than 3 ft. wide, should be left uncovered, so that the slope puddle may afterwards be joined up to it; and this margin should be formed thus:—

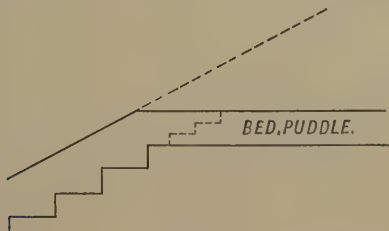


Fig. 25.

So that when the joining up is to be done the layers may overlap and be re-worked into each other. This part of the work requires the greatest watchfulness to ensure soundness.

It is often the case that the thickness of the puddle wall of service reservoirs is made less than that of storage reservoirs proportionally to the height, but it is not very obvious that this is sound practice. It may be said that in the higher bank there are more risks of disturbance; yes, but then the puddle is made thicker in proportion to the height. Of course, if the larger reservoir burst, more damage will be done than if the smaller one were to fail; but this hardly forms a good reason, for the damage done by the bursting of even the smaller reservoir would be sufficiently serious to prevent any risk being incurred on that score. As long as the puddle wall of the larger bank is held up in its place, it has no more force to resist, proportionally to its height, than that of a smaller bank, and the possibility of its not being constantly held up in its place cannot be admitted in the case of the smaller any more than in the larger bank. The only reason of any value why the puddle wall of a large reservoir should be proportionally thicker than that of a smaller one admits a deficiency of practice. It is that in works of magnitude, where large numbers of workmen are employed, it is more difficult to ensure sound work than where the attention of the inspector can be more concentrated; but that only admits the insufficiency of the number of inspectors. And if it be said that there is a difference between the two cases, inasmuch as the service reservoir is generally kept full, while the storage reservoir is subject to fluctuation of level, and therefore of pressure on the puddle wall, this does not seem to touch the question, but rather to be a consideration in the formation of the inner slope of the bank which is to protect the puddle wall.

The bottom should be formed with an inclination from the sides to the centre, and along the centre there should be a channel with an inclination towards the lower end, and from the lower end of this channel there should be a pipe laid to lay dry the bottom of the reservoir—the drain pipe. The level to which the water is drawn off for use is the foot of the slope, some feet above the bottom of the reservoir. The water is drawn off through copper-wire gauze strainers of from 40 to 60 strands to the inch.

For smaller reservoirs, covered over when finished, vertical

walls are built, and the excavated material embanked around them. Sometimes these reservoirs are made circular, but they are difficult to cover economically, although the form itself is economical in respect of wall material, the circle containing a greater area than any other form having the same length of wall.

The best way of covering service reservoirs is by running 14-in. walls lengthwise at distances of from 15 ft. to 20 ft. apart, and turning 9-in. brick arches over the spaces, having a rise of one-fifth of the span, filling up the spandrels with concrete, and covering over the whole with earth 18 in. or 2 ft. in depth. Openings may be made in the walls, either altogether circular or in the form of vertical spaces arched over, so that the wall consists of a series of piers supporting a continuous depth of wall—say 2 ft.—from which the main arches spring. One or more openings in the arches should be made for the escape of air, and a man-hole should be provided, with ladder bars built into the wall, or, which is perhaps preferable, an iron ladder should be provided by which access may be had to the bottom. Ladder bars built into the wall have sometimes been made of cast iron, but at least one serious accident has happened to a workman by the sudden breakage of one of these bars on receiving a blow, and they ought to be of wrought iron.

An overflow weir, capable of passing over the quantity of water that the conduit or the pumping main will deliver into the reservoir, should be placed at the top-water level, and, having fixed upon the greatest height to which the water is to be allowed to rise above this level—say 6 in.—the length of the weir may be found (without going into the particulars of the exact form of the weir) by the general equation $l = \frac{q}{5\sqrt{d^3}}$,

where q =the quantity of water in cubic feet per minute, d =the depth in inches measured from a still head, and l =the length required, in feet.

The sluice valve, by which the water is discharged from the service reservoir, belongs to the main, which forms the first feature in the means of distribution.

A service reservoir acts as a pressure regulator. It would be inconvenient to connect the service pipes of the distribution directly with the main pipe which conveys the water from the storage reservoir; it might be necessary to shut off the water at the reservoir for a day or two for repair of the conduit

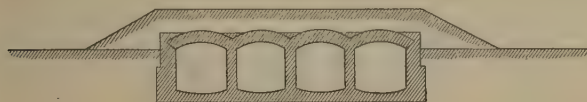
pipe or other purpose, and another inconvenience would be that the pressure due to the height of the column could not be economically made use of; the pressure would be least when most wanted. It is a common occurrence that during the busy hours of the day twice as much water per hour is used as the average hourly quantity, and at those times the velocity in the main would be doubled and the loss of head increased four times, and the serviceable pressure proportionately reduced.

The same thing occurs in the distributing main and service pipes in connection with a service reservoir, but with this difference—that, the velocity in them being less, there is less absolute loss of pressure; doubling the smaller velocity and squaring it for its effect does not produce much absolute loss of head. The excessive loss of pressure at the place where the water is to be used is avoided by interposing a service reservoir in which the flow of water expands over an area, and stands nearly at the same height at all times. This evenness of pressure is a practical advantage, although, no doubt, greater pressure would be obtained without the interposition of the service reservoir if but little water were being drawn from the pipes, and if the main conduit were a pipe laid below the hydraulic gradient. For any purpose for which the water would need to be shut off at the storage reservoir, such as the connection of a branch or the repair of the main, 48 hours would probably be time enough for emptying the main, doing the work, and refilling the main, and in the following remarks it will be assumed that the service reservoir is to hold two days' supply of water.

A system has been in a few cases adopted of making very large service reservoirs, where the ground is sufficiently favourable, and thus increasing the storage capacity of the works materially, the form of the ground in these cases being such as to admit of water being impounded by an embankment on one or two sides of the site, or it may be partly on a third side; but usually there is little choice of situation for a service reservoir, its primary necessity being height, with reference on the one hand to the height of the highest ground to be supplied, and on the other to the height of the storage reservoir, the dimensions of the conduit and conduit pipe being made accordingly. The situation of a service reservoir being thus almost rigidly fixed, it is dug out of the ground for the most part, and the excavated earth embanked round it.

The four essential features of a supply of water by gravitation are the storage reservoir, the conduit, the service reservoir, and the distributing main ; and service pipes. Filtration forms a fifth when it is necessary.

Where the supply of water in bulk may be undertaken in connection with the regulation of flood waters, the question of authority divides itself between the landowner on the one hand, and the local water distribution authority on the other, or perhaps several local authorities in each case. The distribution of the water, and its filtration, if necessary, must be with the water authority, but the service reservoir should be made by the reservoir authority as distinct from the distribution authority, and the latter would take the water from the service reservoir at a price per 1,000 gallons or per million gallons. Near populous places, where there is usually excessive dust and smoke in the atmosphere, water for drinking cannot properly be exposed, and each water



COVERED SERVICE RESERVOIR

Fig. 26.

authority would construct a covered service tank, into which the water would flow from the service reservoir. One service reservoir might suffice for several covered service tanks, which, in some cases, might belong to different local authorities, whether urban, as local boards, or rural, as boards of guardians. Whether one or several water authorities take water from the service reservoir, each would lay a distributing main between the covered service tank and the service reservoir, and the service pipes in connection with it. Adjoining the service reservoir would be a measuring tank for each authority, in which would be the gauge, open to inspection, and so arranged as to give the quantity of water agreed upon from time to time up to a certain maximum.

The use of the service tank at the end of the distributing main would be to receive the excess of water not used, and to contain a day's supply in case of its being necessary to shut off the water at the service reservoir, of which notice could always be given so as to have the tank full. By the continued

record of the state of the service tank it would be seen whether more water were being taken than was used, and it should be a condition of the agreement between the water authority and the reservoir authority that the quantity may be varied from year to year as circumstances require, within a certain maximum. This, no doubt, would throw an onus upon the reservoir authority, but only a proper one, as being the dispenser of the water, and being able to estimate the proper provision to be made for present and prospective needs independently of any estimate made by the sanitary authorities.

Amongst the varying levels at which places to be supplied with water are situated in respect of any storage reservoir, three may be taken for example in which the covered service tanks, A, B, and C, would be at such levels that the highest would be about 100 ft. below the lowest level to which the water in the reservoir would be drawn down, and if the population of the three places or districts be 30,000 in all, there would be required for a full supply 750,000 gallons a day.



Fig. 27.

It would be a moderate view of the circumstances to assume that within an average distance of three miles of the three places a service reservoir might be made on ground sufficiently high to command them all, and that the water may be impounded at the height named within a distance of seven miles from the service reservoir.

The height of the service reservoir in such a case would be at about the mean height between the covered service tanks and the lowest water-level of the storage reservoir. There would then be about 16 ft. or 17 ft. fall per mile in the distributing main, and 5 ft. per mile in the conduit. This would be sufficient if the water were carried along the hill-sides, but if any considerable length of low ground intervened, requiring to be crossed by an iron pipe, the total fall of the conduit could be a little increased and the fall of the distributing mains as much reduced.

Construction.—An open reservoir should be at least 14 ft. or 15 ft. deep. The rays of the sun act through shallow water upon the bottom, and cause the growth of vegetation and insects. If the service reservoir in question were required

to hold 1,500,000 gallons, or 240,000 cubic feet, being two days' supply, and it be made 15 ft. deep, the water area at the mean depth would be 16,000 sq. ft., and if it were made square it would be, at that depth, 127 ft. square. According to the nature of the ground, the slopes may be from $1\frac{1}{2}$ to 1 to 2 to 1. Let it be assumed that they are 2 to 1 throughout, both in the excavation and embankments, and that the top of these is 2 ft. above the top-water level. The reservoir would then be at the top 165 ft. square. The top of each bank may be 6 ft. wide, except on the lower side, where, indeed, the width may be the same and the slope the same, but divided by a level bench of such width that the bulk of earth in the lower bank would, if continued in one slope, make the top width 9 ft. Here we have no puddle trench, and to make the reservoir watertight, the puddle—if puddle be used—is continued across the floor and up the slopes of the excavation, and along the surface of the ground to meet and join with the puddle walls raised in the centre line of the embankments. The floor-puddle may be 2 ft. thick if well made, and the wall-puddle should be at the bottom as thick as will allow the top to be finished 3 ft. or 4 ft. thick, with a gradual reduction in the thickness of 1 ft. in each 6 ft. in height. The surface of the floor-puddle and that upon the slopes, should be covered with broken stone or gravel, not less than 6 in. in thickness, which may be continued up the slopes of the embankment to the top of the reservoir. This hardly forms of itself a sufficient protection of the puddle against the action of the water upon it, but if the interstices are filled with sand it does so. There is but little difference between this and concrete for this purpose. But to provide a surface which can be swept, brick-on-edge paving is added, or two courses of bricks laid flat.

If the water is not to be filtered, it is necessary that it should pass through fine wire-gauze strainers in going out of the reservoir. To hold these and the valves through which the water would pass, a structure of masonry is built.

The strainers are of copper wire, 40 to the inch; but as these, being of so fine a mesh, would too soon require removal if not protected, strainers of a mesh less fine, about 20 to the inch, are placed in front of them, and a double set of grooves is provided for these, so that before one set is removed the other may be inserted. The screens are removed and washed as occasion requires, being for that purpose and for strength

made in small rectangular frames of wood, inserted in a larger frame which can be lifted out bodily, these being set one upon another to the height required.

The top soil, before the excavation is begun, would be taken off and reserved outside for soiling the slopes of the banks. The tops of the banks would be formed as footpaths, with gravel or any clean material, and formed with a transverse inclination away from the reservoir.

The outside of the puddle wall should be protected from rats by a facing of concrete, or else the earth may be mixed with lime in the proportion of about 10 to 1, and watered for a yard in width outside the puddle wall, and this should be extended along the surface of the ground for some part, if not the whole, of the seat of the outer part of the embankment.

In the case of a high embankment, as of a storage reservoir, this protection is not necessary, because the distance from the outside to the puddle wall is too great to be burrowed, but in small service reservoirs the puddle should be protected.

In many situations where it would be proper to construct a service reservoir, it would be difficult to procure clay for puddle within any short distance, and it might, at the same time, be easy to procure materials for concrete. In that case the reservoir might be made watertight, without puddle, by the use of concrete made specially with that view, that is, with plenty of filling, both sand and lime or sand and cement; but if good blue lias lime can be procured at a less price than cement, it will be preferable to use lime, the quantity being increased in the ratio that the price of cement bears to that of lime; and the thickness of concrete may be less than that required of puddle; and, reckoning the thickness of puddle and its covering together, this is a considerable reduction of the thickness of materials required for the bottom of the reservoir, and saves excavation to the extent of about half the difference of the cubic contents.

If, however, there is the slightest reason to fear a movement of the ground, however slight or partial in its extent after the reservoir has been made and filled, puddle will be preferable to concrete as being less rigid. At the same time the longitudinal tenacity of concrete may be a favourable property in the opposite respect. The earth would be wheeled out, or carted out, and put into the banks in thin layers not more than 6 in. in thickness, and consolidated by hard ramming, and, if sand, watered at the same time.

A sand bank made in wet weather is very much superior to one made in dry weather.

If the banks be wholly of sand the puddle walls should be of greater thickness than is otherwise necessary, unless they be faced with clayey material, or with limed earth to prevent the moisture of the puddle being absorbed by the banks.

Service reservoirs, larger and deeper than the one of this example, have been made on ground containing nothing but sand, and with the strength-dimensions here stated, stand firm and watertight; but ground containing sand, clay, and stone shale mixed is preferable, if the clay be not in excess. Instead of the expensive brick paving of the inside of the reservoir, a facing of Portland cement on concrete would be almost equally good, and considerably less expensive.

A particularly good fence is necessary round the foot of the slopes. Post and rail is good enough for a storage reservoir, but not for this, and the fence should be far enough from the foot of the slope to allow a cart-way all round, and a piece of land should be enclosed near the outlet of the reservoir for stores and other purposes.

The suggestions made by the original author in this chapter still hold good when this method of construction is adopted, but modern practice tends to the use of ferro-concrete almost exclusively for service reservoirs, unless of very large size. In general, this form of construction, which has everything to recommend it for the purpose, consists of laying the floor of the reservoir on the usual system of slab construction, of forming the walls on the buttress and slab principle, with the roof built on the slab and beam system and supported on a series of reinforced columns. It is not necessary to here dilate upon this work, because it is a specialised branch of engineering which has been applied with success and economy to the construction of service reservoirs often of considerable size, while the method is now a favourite one for the construction of water towers. In the latter connection, however, mention should be made of the sectional cast-iron tank if the work has to be carried out rapidly or in localities where concrete construction would be difficult. By a system of standardised rectangular flanged plates, together with corner and angle plates, all having machined flanges and bolt holes drilled to jig, large tanks can be built up with extraordinary rapidity. The method is, admittedly a little costly, but it is so simple and durable that further comment is needless. When necessary such tanks may be lined with concrete.

CHAPTER XIV

PRESSURE, AND ITS EFFECT IN PIPES

THE force and mode of action of water under pressure along and at the end of a main pipe may be worth considering in connection with its storage in reservoirs such as have been described. The pressure of still water is the weight of a vertical column of it above the place where the pressure is measured; but still water does no work, and when the column is in motion the pressure is less. The weight of water is as follows, the foundation being the Troy grain, 5,760 of which used to make a pound weight; but this pound not being satisfactory for general purposes in England, the pound weight was increased to 7,000 of those grains. An Act of Parliament made a gallon of distilled water at the temperature 62° F. to weigh 10 lb., or 70,000 grains, in air of the density produced by a pressure equal to the weight of a column of mercury 30 in. high; and established also, from experiments which had been made by a commission, that a cubic inch of distilled water at the temperature and pressure above mentioned weighs 252.458 grains. A cubic foot therefore weighs $252.458 \times 1,728 = 436,247$ grains, or $\frac{436,247}{7,000} = 62.321$ lb. In

half a dozen different tables of the weight of mercury compared with that of water, six different values may be seen—viz., 13.56, 13.568, 13.57, 13.58, 13.596, and 13.6. When tables differ, what is the proper weight? The difference seems to arise from comparing the weight of mercury at one temperature with that of water at another, in some cases, and in other cases in taking water sometimes at the temperature 39.2° F., when it is at its greatest density, and at other times taking it at the common temperature of 62° F. This latter is the more useful for ordinary purposes. At 62° F. mercury is 13.596 times heavier than water. The pressure of the atmosphere, then, is the same as a column of water

$\frac{30 \times 13.596}{12} = 33.99$ ft. high, or 34 ft., and the corresponding

pressure per square inch is $\frac{34 \times 12 \times 2.52 \times 458}{7,000} = 14.71$ lb. When

the mercury rises to 30.5 in. the pressure is 15 lb. nearly per square inch. But as the weight of the atmosphere is often less than 30 in. of mercury, and sometimes only 28.5 in., it is the minimum pressure which should be reckoned upon in practice in order to guard against failure of action at all times.

This is $\frac{28.5 \times 13.596}{12} = 32.29$ ft. of water, and the corresponding

pressure per square inch is 13.97 lb. or 14 lb. As the head of the column of water is open to the atmosphere equally with the point at which the pressure is applied, this pressure is "above the atmosphere," and although there may be a little difference between the pressures of the atmosphere at the two ends of the column, measured at the same instant, if the pressure reckoned upon be that of the minimum there will be no failure of effect.

The weights above mentioned relate to water without admixture of other matter; common water contains in solution heavier matter than water itself, derived from the ground through or over which it runs, and in practice a cubic foot is taken to weigh $62\frac{1}{2}$ lb. and to contain $6\frac{1}{4}$ gallons.

In the case of a pipe conveying water from a reservoir to the place where it is to be used, the column of water divides itself into two portions, and the pressure at the lower end of the column is according to the height of the lower portion. The upper portion, being that part of the whole column between the top of the virtual column and the level of the water where it meets the atmosphere, is the head of water, which feeds the column as fast as it descends by the force of gravity. In a pipe of any given diameter, as 1 ft., the resistance of the sides of the pipe to the motion of the water is proportionate to its length, for it is proportionate to the area of surface with which the water runs in contact; secondly, it is proportionate to the velocity of the water, for it is proportionate to the number of particles of water in contact with a given length of pipe, as 1 ft., during any given time, as one second. But the head of water required to supply the force of which the column of water is robbed in overcoming this resistance must be as the square of the velocity, for not only is it thus simply proportionate to the velocity with which

the water moves, but it must be replenished as fast as it runs away. If the length of pipe in contact with the water during one second be twice as great in one case as in another, then there will be twice the resistance due to that cause, requiring twice the head to overcome it, and at the same time the water will run away from the head twice as fast, and require twice the length of pipe full of water to replenish it in the same time.

The head, therefore, is as the square of the velocity. Now as the altitude of the proper head cannot be increased, but remains at the same level, nearly, whatever the velocity in the pipe may be, the only way in which the head can be increased is by taking from the length of the real column the height which may be necessary to give the required velocity—increasing the head at the expense of the real column, and thereby reducing its effective pressure. The same effect may be shown by substituting actual pressures for heads of water in a horizontal pipe, such as a pumping main. At a point on a line of main let three sections of it be marked off, of equal length, say 1 ft., in the direction in which the water flows, and let these sections be numbered 1, 2, and 3 from the point of observation. From the same point let three other sections be marked off, of the same length, in the direction from which the water comes, and let these be called A, B, and C, and let them be bodies of water. Let the diameter of the pipe be such that each body of water requires a pressure of 1 lb. to be given to it to overcome the resistance of each of the sections 1, 2, and 3. Then let three operations be performed :

- | | | |
|---|----------|---|
| 1. A moves through section 1, requiring a pressure of 1 lb. | .. | 1 |
| 2. A and B move through sections 1 and 2, A requiring 2 lb. and B 2 lb. | | 4 |
| 3. A, B, and C move through sections 1, 2, and 3, A requiring 3 lb., B 3 lb., and C 3 lb. | | 9 |

Then if each operation is performed in the same time, as one second, the pressure required is as the square of the velocity.

If feet vertical of water be substituted for pounds pressure, the illustration serves equally for a head of water.

In respect of diameter: in pipes of different diameters the resistance is inversely as the diameter. The resistance is, indeed, directly as the area with which the water runs in contact, which in the same length of pipe is as the circumference, and therefore as the diameter, and a diameter of two offers twice as much resistance as a diameter of one; but the distance at which it acts—the radius of the pipe—is twice as

far removed from the axis (the point in which there is no resistance), and this, be it remarked, in two opposite directions, so that the effect of distance is as the square of the radius, and therefore of the diameter; the resistance being direct in its action, and the distance at which it acts inverse, the result is that the resistance is inversely as the diameter.

These several forces and resistances are formulated thus :—

Let v =velocity per second.

l =length of pipe.

d =diameter.

h =head of water.

$$\text{Then } v = \frac{cdh}{l}$$

in which c is a constant multiplier deduced from experiments to be 2,500 when all dimensions are taken in feet. This is the value of c according to Eytelwein's deduction for long pipes. The velocity, then, in feet per second is

$$v = \sqrt{\frac{2,500dh}{l}} = 50\sqrt{\frac{dh}{l}}$$

and the diameter is $d = \frac{v^2 l}{2,500h}$. This is the usual requirement

in the case of a pipe to convey water from a reservoir to the place where it is to be used, for in such case the height and length are fixed, and the velocity must be limited so that at its maximum it may do no harm to the pipe by its violence, and so that branches derived from it may be duly filled, and so that a sufficient working pressure be given.

When the pipe is several miles in length the rule is converted into a form expressing the head of water in feet per mile by dividing 5,280 by 2,500, in which l is eliminated, and $h = \frac{2 \cdot 11 v^2}{d}$. If the velocity be made 3 ft. per second, $h = \frac{19}{d}$,

and the pressure of water running with that velocity would be, at the distance of one mile from the reservoir, less than that due to the whole height of the column by $\frac{19}{d}$.

It might seem that the pressure would be that due to the whole height less such a head as is required to produce the velocity considered as falling water merely, which would be $\frac{v^2}{64}$; but although that is so in open streams, which are in

train, it is not so in pipes under pressure. In this case, as in the other, there is a certain gradient line, which, if drawn straight between the head of water in the reservoir and the height due to the pressure at the end of the pipe, is the gradient which that pipe would necessarily take if it ran full, but only full, that is without pressure on the highest part of its circumference; and the pressure at any part of the length of a pipe is that due to the vertical height between the actual position of the pipe and that gradient, which is the "hydraulic gradient." There being thus a gradient line in all conduit pipes, there must be a length at which there would be no pressure, supposing the pipe to be prolonged so far from the reservoir, and there the water would simply run out of the end of the pipe with the velocity due to the gradient. This length is found as follows:—

Substituting H , the whole head, for h , the head which is due to the velocity v , $l = \frac{2,500dH}{v^2}$. For example, let $d=2$ ft.,

$H=100$, $v=3$, then $l = \frac{2,500 \times 2 \times 100}{9} = 55,555$ ft. Thus at

the distance of 10 miles or so there would be no pressure in a pipe 2 ft. diameter and 100 ft. below the reservoir, and it could not carry the water farther than 55,555 ft. with a velocity of 3 ft. per second; beyond that distance the velocity would diminish, unless an actual gradient were immediately given to the pipe equal to the hydraulic gradient; but that being done, the same velocity would continue to any distance, if the diameter remain the same; the stream of water through the pipe would then be "in train," the forces acting upon it being balanced by the resistances, and it would flow under the same conditions as a river. But if pressure were required, there would need to be either a greater head, a shorter length, or a larger pipe. The conditions to be determined beforehand are (1) the quantity of water required, (2) the pressure required, (3) the height at which it is to be supplied above a fixed datum level, and (4) the height of the reservoir above the same datum.

In one of the examples previously referred to, the quantity of water for supply was found to be 3,000,000 gallons a day, after leaving to the stream one-third of the total available quantity. If the distance at which this quantity were required to be delivered were 5 miles, and the height of the storage reservoir above the service reservoir in which the

water would be delivered were 150 ft., the following conditions would ensue. The quantity per second corresponding to 3,000,000 gallons a day is $\frac{3,000,000}{540,000} = 5.55$ cu. ft. If the velocity be limited to $2\frac{1}{2}$ ft. per second, the diameter would be $d = \sqrt{\frac{5.55}{2.5 \times .7854}} = 1.68$ ft., say 21 in. The head would be $h = \frac{5,280 \times 5 \times 6.25}{2,500 \times 1.68} = 39.29$ ft., say 40 ft. The effective head would be $150 - 40 = 110$ ft., and the pressure per square inch $\frac{110 \times 62.5}{144} = 47.7$ lb. at the end of a 21-in. main 5 miles long.

Upon a line of conduit pipe such as this there may be taken off one or two branches. Say that one-sixth of the water is wanted at three miles' distance from the reservoir, and another sixth at four miles, leaving 2,000,000 gallons per day to go to the far end. If the same velocity were preserved throughout, the square of the diameter of the pipe would be reduced one-sixth at the first branch, and one-third at the second. Thus the first length of three miles being 21 in. diameter, the second length of one mile would be 19.18, or, say, 20 in.; and the third length of one mile 17.14, or, say, 18 in. Each branch takes off 500,000 gallons a day, and, preserving the same velocity, the diameter of each would be

$\sqrt{\frac{(21)^2}{6}} = 8.51$ in., or, say, 9 in. The effective head in the

main conduit pipe at the point where the first branch is derived would be found by deducting the loss of head due to its distance from the reservoir, three miles, which would be

$\frac{3 \times 40}{5} = 24$ ft., from the difference of altitudes of the reservoir

and the branch above the datum. For the second branch the deduction would be rather greater per mile, the diameter being less, while the velocity is the same, for the head is inversely proportionate to the diameter, according to the formula $chd = v^2 l$. Thus if the loss of head due to a velocity of $2\frac{1}{2}$ ft. per second in the 21-in. pipe be 8 ft. per mile, it would be in the 20-in.

pipe $\frac{21 \times 8}{20} = 8.4$ ft. per mile, and in the 18-in. pipe $\frac{21 \times 8}{18} = 9.4$

ft. per mile. Adding together the losses of these three lengths—viz., for the first three miles, 24 ft., for the fourth mile, $8\frac{1}{2}$ ft., and for the fifth mile, $9\frac{1}{2}$ ft. the total loss at the end of

the five miles would be 42 ft., instead of 40 ft., as it would be if the full diameter were continued to the end, and the quantity delivered were 3,000,000 gallons a day.

Cast iron is the material for water-pipes, from 3 in. to 3 ft. 6 in. diameter. They have been cast as large as 44 in. and 48 in., but there are several reasons of convenience and economy which make it desirable to limit the diameter of cast pipes to about 40 in. or 42 in. If the quantity of water to be conveyed is such as to require a larger diameter, it would probably be found that either two cast pipes, or one pipe built up of rolled plates, would be preferable. They are sometimes made smaller than 3 in. ; but the expense of laying constitutes a considerable part of the whole expense of small pipes, and it is not much more for 3-in. than for 2-in. pipes.

The metal of which pipes are cast is not so strong as that put into girders and some other structures. The tensile strength of this metal is about eight tons per square inch ; but that put into pipes is probably not more than seven tons per square inch for many of the pipes of every set, and it is the strength of these upon which the success of the work depends. Whatever the quality and ultimate strength of the metal may be, the working strain should not exceed a determinate part of it, arrived at in two stages of the process of calculation.

In the first place, every pipe is subjected to an actual water pressure before it is laid. If the pressure be applied steadily to represent the effect of a head of water, it may be as much as one-half of that which the metal would bear before breaking ; but if, while under pressure in the proving press, the pipes be struck with a hammer to represent the jarring and blows to which they are subject in the ground, the pressure applied should not be more than one-third of the ultimate strength of the metal.

The proof strain of the pipes in the press should not, on the one hand, be so great as to injure the strength of the metal, which would probably happen if more than half the pressure due to the ultimate strength were applied ; but, on the other hand, the actual strain produced in the metal should be sufficient to prove the soundness of the casting, and the quality of the workmanship.

But between the time of proving and being laid in the ground the pipes are subject to a variety of accidents which tend to produce defects, some of which may be undiscover-

able; and, moreover, the arrest of the motion of the water passing through them produces a greater strain than that due merely to the height of the column of water. A complete stoppage of the motion could not be suddenly effected, but there is no doubt that the shutting of stop-cocks, in a system of piping, throws additional pressure on some parts of it; and the working pressure, due to the head of water, should not be more than some determinate part of that applied in proving the pipes—say one-third of the pressure applied in the first-named manner, or one-half applied in the manner secondly named; so that, in calculating the thickness of metal, the steady pressure due to the head of water should not exceed one-sixth of the ultimate strength of the metal. If this be seven tons, or, perhaps, not more than 15,000 lb. per square inch, the strain produced by the head of water should not exceed $\frac{15,000}{6} = 2,500$ lb. per inch.

If H = the height of the column of water in feet, the pressure per square inch on the internal surface of the pipe is $\frac{62\frac{1}{2}H}{144} = .434H$.

This is a radial pressure acting on the whole circumference of the pipe equally all round, except that it is less on the top of the pipe than the bottom, by the pressure due to the diameter; but this may be neglected, and its consideration may, in fact, be done away with, by measuring the height of the column of water from the bottom of the pipe. But, although the pressure is radial, and the circumference receives a nearly equal pressure per square inch all round, the component part of the pressure which constitutes the force tending to tear the metal asunder, acts in a direction tangential to the circumference at any point, and at right angles to the radius; and as all the radii are alike in a circular pipe, the force tending to burst the pipe is proportionate to the radius, and is measured by it multiplied into the pressure due to the head of water. For calculation of the thickness of metal, it answers all purposes to take one inch of the length of the pipe.

The whole outward pressure in pounds, tending to separate one-half of the ring from the opposite half, is $.434H$ multiplied into the diameter in inches, and this force is restrained by the two sides of the pipe. As it is only necessary to take into account one side, the strength may be calculated from the radius instead of the diameter. Let $.434H = p$ = the pressure

of the water in pounds per square inch; r =the internal radius of the pipe in inches; t =the thickness of the metal, and s =the strain per square inch of the metal acted upon, to which it is subject under the pressure p and radius r , then the following quantities should be equal to each other, viz.

$pr=st$, and $t=\frac{pr}{s}$. But here is an anomaly; for the part of

the metal which the pressure acts upon with the force pr is infinitely thin. Beyond the face, the metal is strained less and less as the radius increases.

The following remarks depend upon the truth of the law that the extension of a material per unit of its length, strained by an elongating force, is proportional to the force applied. The extension of cast iron, under all strains up to that which breaks it completely, is not exactly proportional to the force applied; but it is so within the limits of the smaller strains to which the metal is subject in water-pipes.

The strength with which any portion of the metal restrains the force which tends to tear it asunder, is measured by the extension of the metal per unit of its length within the limits of its proper elasticity, and both the force and that extension diminish as the radius increases. The metal at the back of the pipe can only assist the strength of that at the internal face, in proportion to the rate of extension which the force within the pipe produces in it; and the rate of extension at any part of the thickness of the metal is inversely proportionate to its distance from the centre of the pipe.

The magnitude of the bursting force is itself proportional to the internal radius, and is confined to it, and its proportional stress at any part of the thickness is inversely as the distance at which it acts; and as the rate of extension is also inversely as the same distance, therefore the metal, at any part of its thickness, restrains the bursting pressure with a force which is inversely proportionate to the square of the radius at that part.

With an internal radius of r , a thickness of r , and a force of r , the back of the pipe is strained with a force of $\frac{1}{2}$, and as the length of its circumference is twice that of the internal circumference, the extension due to a force of r would there be $\frac{1}{2}$; but as the force is only $\frac{1}{2}$, the strain there is $\frac{1}{2}$ of $\frac{1}{2}$, or $\frac{1}{4}$; that is, it is inversely as the square of the radius.

In a 10-in. pipe, 1 in. thick, the internal radius is 5 in. and the external radius 6 in., and with any given pressure

of water the back of the pipe is strained less than the face in the ratio $5^2 : 6^2 = 25 : 36$.

For the sake of showing the effects in an extreme case, we may take the metal to be still thicker. The distance at which the force acts directly on the metal is limited to a radius of 5 in. As the force does not act directly upon the metal beyond the face, but indirectly through the interposing metal, it is diminished as the radius increases. At 6 in. radius the force is $\frac{5}{6}$ ths of whatever it is at 5 in. ; at 8 in. radius it is $\frac{5}{8}$ ths, and so on ; and the effect of this diminishing force is further reduced in the same ratio by the distance at which it acts, so that at 6 in. from the centre of the pipe there is a force of $\frac{5}{6}$ ths acting at a distance of $1\frac{1}{5}$ th ; at 7 in. a force of $\frac{5}{7}$ ths acting at a distance of $1\frac{2}{5}$ ths, and so on.

Whatever strain per square inch it may be thought proper to subject the metal to at the interior of the pipe, that at the back will be in the following proportion. Let the maximum strain be assumed to be 2,500 lb. per square inch, then

At 5 in. radius (face of the metal)	$\frac{2,500 \times 5}{5} = 2,500 \text{ lb.}$
„ 6 in. „ „ „	$\frac{2,500 \times 5}{6 \times 1\frac{1}{5}} = 1,736 \text{ lb.}$
„ 7 in. „ „ „	$\frac{2,500 \times 5}{7 \times 1\frac{2}{5}} = 1,276 \text{ lb.}$
„ 7½ in. „ (middle of the thickness)	$\frac{2,500 \times 5}{7\frac{1}{2} \times 1\frac{1}{2}} = 1,064 \text{ lb.}$
„ 8 in. „ „ „	$\frac{2,500 \times 5}{8 \times 1\frac{3}{8}} = 976 \text{ lb.}$
„ 9 in. „ „ „	$\frac{2,500 \times 5}{9 \times 1\frac{4}{9}} = 772 \text{ lb.}$
„ 10 in. „ „ „	$\frac{2,500 \times 5}{10 \times 2} = 625 \text{ lb.}$

The mean strain (M) of the whole thickness is not $\frac{2,500+625}{2}$, but is the strain (S) at the face of the metal multiplied into the square of the internal radius, and divided by the mean of the squares of the internal radius (r) and the external radius (R).

$$\text{Thus } M = \frac{2r^2S}{R^2 + r^2}$$

In the case above stated it is $\frac{2 \times 25 \times 2,500}{100 + 25} = 1,000 \text{ lb. per square inch.}$

The force producing these strains is pr = the pressure per

square inch, multiplied into the internal radius of the pipe in inches. In order that the inner portion of the metal be not overstrained, it is necessary to extend the thickness far enough to comprehend a restraining force equal to that exerted on the inner portion of the metal, and its limit is found at a radius the square of which bears the same relation to the square of the internal radius as the force exerted at the face of the metal bears to that exerted at the back, and in very thick pipes the mean strain per square inch of the whole thickness differs widely from the arithmetical mean between the strains at the internal and external radii, because the strain at every part of the thickness is as the square, inversely, of its distance from the centre of the pipe, but at all distances which lie within the thickness of ordinary cast-iron pipes the mean strain would not differ much from the arithmetical mean between the inner and outer strains.

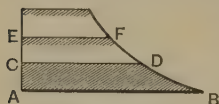


Fig. 28.

In the diagram Fig. 28, AB represents the strain per square inch at the face of the metal, and CD, EF, etc., the strains at the several distances C, E, etc., from the centre of the pipe, the corresponding strains being inversely as the squares of those distances, in pounds per square inch, produced by the force pr .

There are many short rules for the thickness of metal in cast-iron water-pipes, and they vary much in the results obtained from them. They may be compared with the principles above stated. Mr. Molesworth, in the well-known *Pocket-book of Engineering Formulæ*, gives the rule,

$t = .000125pd + .37$ for pipes less than 12 in. diameter; or,

$t = \text{the same} + .50$ for pipes from 12 in. to 30 in. diameter,

t being the thickness of metal, p the pressure of water per square inch, and d the diameter in inches. To adopt a uniform notation, let the radius (r) be taken instead of the diameter, then

$t = .00025pr + .37$ for pipes less than 12 in. diameter; or,

$t = \text{the same} + .50$ for pipes from 12 in. to 30 in. diameter.

Mr. Neville, in *Hydraulic Tables, Co-efficients, and Formulæ*, gives the rule for pipes cast vertically,

$$t = .0016nd + .32,$$

and for pipes cast horizontally,

$$t = .0024nd + .33,$$

in which n is the number of atmospheres of pressure, = the head of water in feet divided by 33. Adopting, likewise, r instead of d , and converting n into pounds per square inch (p), the rule would be, for pipes cast vertically,

$$t = .0002234pr + .32,$$

and for pipes cast horizontally,

$$t = .000335pr + .33$$

But Mr. Neville remarks that, in practically applying these formulæ, the value of n should always have ten added to it. He mentions a formula adopted by M. Dupuis, the engineer of the Paris Waterworks, which is $t = (.0016n + .013)d + .32$.

Taking in this case also the radius instead of the diameter, and converting n into pound per square inch, the rule would be $t = (.0002234p + .026)r + .32$.

Professor Rankine, in his *Manual of Civil Engineering*, gives the rule $\frac{t}{d} = \frac{H}{12,000}$, in which H is the head of water in feet. This would be equivalent to

$$t = .000192pd$$

or,

$$t = .000384pr$$

But Dr. Rankine remarks that shocks from without cause the thickness of cast-iron pipes to be often made considerably greater than that given by the above rule. The following empirical rule, he says, expresses very accurately the limit to the thinness of cast-iron pipes in ordinary practice—viz. that the thickness is never to be less than a mean proportional between the internal diameter and $\frac{1}{48}$ in.

Mr. Thomas Box, in *Practical Hydraulics*, gives as an empirical rule,

$$t = \left(\sqrt[10]{d} + .15 \right) + \left(\frac{Hd}{25,000} \right)$$

t , d , and H being the same expressions as those before stated, and in the same terms.

It is to the investigation of Professor Peter Barlow that we are indebted for a solution of the difficulty of apportioning to cast-iron pipes a thickness properly proportionate to the strain upon them from internal pressure. Barlow's rule is

$t = \frac{pr}{c - p}$, in which t , p , and r are the same expressions as those used above, and c = the cohesive strength of the metal per

square inch, which is assumed above to be 15,000 lb. before it breaks, but only 2,500 as a maximum working strain on the inner portion of the metal, and it is only metal of good quality which it would be safe to subject to even this degree of strain—in metal, that is to say, produced from “mine” or native ironstone; with any considerable admixture of inferior metal the pipes would probably not bear straining nearly so much. Professor Barlow’s investigation did not extend to the minutiae of foundry necessities, such as increase of thickness for unavoidable defects in casting, or the difficulty of making very thin castings; but it established a true basis around which these practical requirements gather, and to which they are more or less wisely added.

To compare these various rules, let each of four pipes—viz., 10 in., 14 in., 20 in., and 30 in. diameter—be subject to the same pressure, viz., 174 lb. per square inch (400 ft. head).

Diameter,	10 in.	14 in.	20 in.	30 in.
p^*	870 lb.	1,218 lb.	1,740 lb.	2,610 lb.
	t	t	t	t
Barlow	·374	·520	·748	1·12
Rankine	·334	·468	·668	1·00
Molesworth	·588	·804	·935	1·15
Neville	·674	·815	1·02	1·38
Dupuis	·644	·774	·969	1·29
Box	·569	·673	·805	·986

According to Professor Rankine’s rule to limit the thinness of pipes to a mean proportional between the internal diameter and $\frac{1}{48}$ in., that would make the least thickness $t = \sqrt{0.0208d}$, and for

10 in. pipes would be	·456
14 in. “ “	·539
20 in. “ “	·645
30 in. “ “	·790

The thicknesses above stated by Mr. Neville’s rule are those of pipes cast vertically. The increased thickness required by his rule for pipes cast horizontally is due to remediable defects, and we must allow to all the other formulæ that they demand strictly proper workmanship, and make no allowance for defects which are not absolutely unavoidable; moreover, casting pipes vertically, of nearly all sizes, is be-

coming more and more the custom. It may be remarked upon Mr. Neville's rule that it is not necessary to add 10 to the value of n (although this is done in the above table) unless it be for very small pressures.

For facility of application, and near agreement with actual practice, Box's rule is preferable to any of the others given above.

While cast-iron pipe still remains as it did when Mr. Slagg made the foregoing remarks upon it, the most serviceable form of pipe for waterworks distribution, a good many improvements have been made since that time. One of the most recent is the introduction of *spun iron pipe*.

Pipe founding is quite a special business and is in the hands of a comparatively small number of firms. The pipes are generally cast vertically. Various difficulties attach to the process, and although such pipes are quite serviceable in practice they by no means approach perfection. To produce better pipes the de Lavaud process of centrifugal casting has been taken up in this country by the Stanton Company, the product being known by the name of Spun Iron Pipe. Pipe founding by this method calls for special machinery, which essentially consists of a rotating steel mould. A pipe extending through the mould starts delivering molten metal at the socket end, and being automatically traversed backward, deposits molten metal all along the mould, which is rotating meanwhile. At the end of the traverse the pipe is complete. This method of pipe founding has met with considerable success, the pipe produced being strictly to dimensions and some 25 per cent. lighter than is practicable by the vertical method. The metal also due to the centrifugal action of the cast is considerably stronger, and incidentally easier to machine.

Joints.—Both turned and bored and spigot and socket pipes have certain inherent disadvantages, amongst which may be mentioned the difficulty of breaking joint if necessary, and of getting round curves with standard straight pieces. To get over these shortcomings a new system of jointing which has been introduced is worthy of notice. Known as the Victaulic, the essential details are shown in Fig. 29A. The collars at the ends of the pipe are machined at a , and over the intervening space b is placed a ring of rubber of "C" section. A metal ring c , which is a simple casting made in two, three, or four pieces, as the case may be, holds the ring

in position. The actual watertightness of the joint depends, in common with other hydraulic packings, upon the water pressure forcing the two lips on to the pipe ends—that is, it is a self-acting packing. It is quite easy to see that this joint

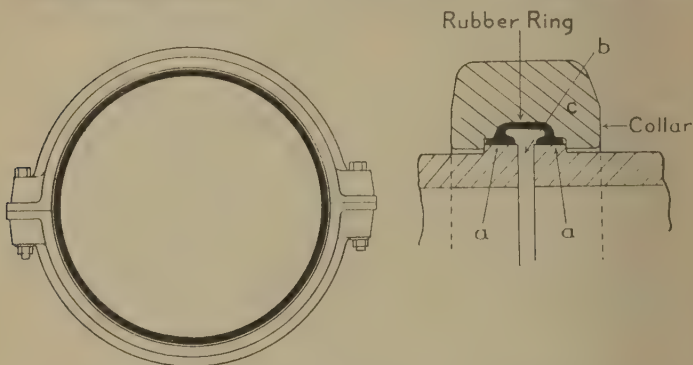


Fig. 29A.

is possessed of considerable flexibility, it readily permits of expansion and contraction and of straight lengths being laid, when necessary, to an appreciable curvature. Moreover, the

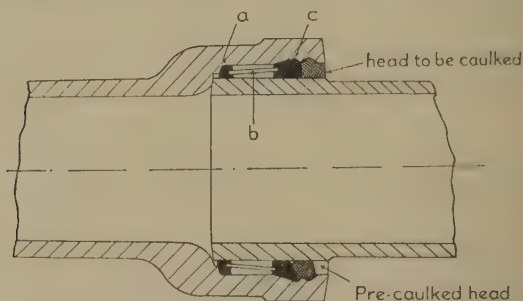


Fig. 29B.

jointing can easily be done in confined spaces. The joint is stated to cost little or no more than the orthodox forms.

Pre-Caulked Joints.—This type of joint is in extended use in America in connection with small-bore water mains. It embodies the well-tried lead joint, but overcomes the troubles attendant upon the making of such joints in narrow trenches. The idea is made clear in Fig. 29B. This joint is

made, at the pipe foundry, about a mandrel somewhat larger than the spigot end of the pipe which it represents. With the pipe stood vertically in a rack, the braided hemp *a*, a ring of wedges *b*, two more braids of hemp *c*, and the final filling of lead are placed. The lower half of the joint is then caulked. In laying, the next pipe end is pushed home in this partly made joint and the top half thereof caulked in the usual way, the bottom or pre-caulked portion merely requiring to be touched up with a special tool. Any deflection of the joint due to faulty alignment merely tends to tighten it, but the principal advantage is obviously the elimination of molten lead having to be poured at the time of laying.

Small Cast-Iron Pipes.—It should be noted that this pre-caulked joint is used mainly with small pipes from $1\frac{1}{4}$ in. to

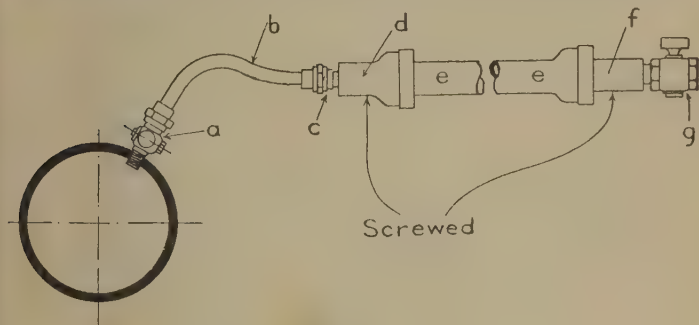


Fig. 29c.

6 in. bore. In British waterworks practice pipes below 2 in. bore are not used, and those less than 3 in. bore rarely met with. Small service connections are invariably made of either lead pipe or screwed steel pipe. The first is costly and the second corrosive, and the American system of using small cast-iron pipes for services, etc., has much to commend it. Arranged as shown in Fig. 29c, a service connection would commence with a cock or connecting ferrule *a*, and a standard form of lead gooseneck *b*, with a flange union *c*. This gives the necessary flexibility to the connection. There is a screwed cast-iron connection *d*, the required number of cast-iron pipes with pre-caulked joints *e*, a screwed make-up piece *f*, and stop-cock *g*.

The arrangement, which makes use of cheap standardised

fittings, avoids both wiped lead joints and any joints being made on the site, other than screwed ones, and finishing off the pre-caulked ones.

A special type of flange joint, which does away with the use of jointing material and incidentally permits of considerable flexibility of alignment, is that shown in Fig. 29D.

It is really a combination of the metal to metal turned and bored joint (drive pipe) and the flanged joint, the actual contact between the pipes being metal to metal, and the pipes themselves held together by bolts passing through lugs.

Essentially suitable for small lines this class of pipe is made in sizes from 2 in. bore to 18 in. bore, and, after testing and coating, the socket or female end is machined to a taper

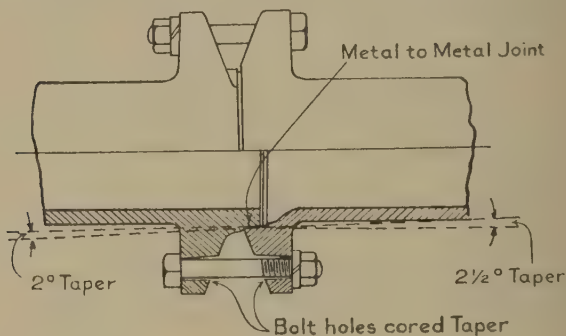


Fig. 29D.

of $2\frac{1}{2}^\circ$ and the spigot or male end to 2° . The joint therefore becomes in effect a wedge joint, and the $\frac{1}{2}^\circ$ difference in taper permits of settlement, expansion, and contraction being taken care of.

Straight pipes of this class can be laid to a radius of 100 ft. As no jointing material is required, and no great care necessary in laying, this class of pipe is very easily set in wet situations, and owing to its flexibility can be used for submerged pipes under most conditions, so doing away with the use of colsty ball and socket joints

Steel Pipes.—While cast iron has been extensively used in the past for the main supplies of our cities steel is taking its place, and is almost invariably used on water-power lines except for small jobs and low-pressure work. For medium

pressures riveted steel pipe is now largely employed. The construction of these pipe-lines aims at providing for as much of the riveting as possible being done in the shops. An instance of a large steel main job is seen in the recently constructed Tansa line for Bombay. A British contract throughout, the plates were as far as possible standardised and shipped to Bombay with edges planed, corners scarfed, and rivet holes drilled to template. Each plate weighed roughly a ton. At a specially constructed factory in India the plates were fabricated into pipe lengths 57 ft. 4 in. long, each weighing over 8 tons; they were transported on special cars over a special line of railway, and put into position by cranes on the job. The average time for the unloading and fitting in position was 12 minutes, and this, together with the fact that the joints which are slip joints came together with almost invariable precision, speaks much for the organisation of the fabrication work.

This continuous method of pipe construction meets most requirements, but for high-pressure work steel-flanged pipes are frequently called for. In this connection it should be mentioned that riveted steel pipes are made when occasion arises with flanged joints, various types riveted and loose being employed.

What are specially referred to now are welded and weld-less (Mannesmann) pipes, while a comparatively recent departure is the reinforced pipe. This is so constructed that the barrel of the pipe is reinforced with high tensile steel bands shrunk on, a method whereby a pipe of great strength may be constructed with a comparatively thin barrel.

Concrete Lined Pipes.—As before stated, while cast iron has the virtue of resistance to corrosion it possesses no great strength, and large pipes are apt to become abnormally heavy. Hence the extended use of steel. With steel, however, one is always up against the corrosion problem, and the lining of pipes with concrete by the centrifugal method is really a development of centrifugal casting, previously described. Known as the Hume centrifugal process, it involves, as its name implies, the formation of the lining of the pipe in revolving moulds. The concrete, of course, has to be of a fine quality and special plant is required for mixing and handling it, but the result with both steel and iron is highly satisfactory, half an inch of concrete giving a smooth non-corrosive bore. Pipes for the new Birmingham waterworks

line were made on this system, the steel shell being $\frac{3}{8}$ in. thick, and the concrete lining $1\frac{1}{4}$ in. thick.

Valves.—In respect of valves the waterworks engineer is well provided for by the better-class waterworks iron-founders. For most general purposes from 2 in. to 24 in. or thereabouts, the sluice valve fulfills all reasonable requirements, but in this simple mechanical contrivance there is considerable difference between a first-class article and one built to a price. With modern machining methods valve makers are able to produce a high-class article made with precision, so that the parts are interchangeable; and with accurately machined bronze faces and spindles it will stand up to severe usage. Apart from watertightness when closed, the primary essential of a valve is that it shall be easily workable at any time, perhaps after a

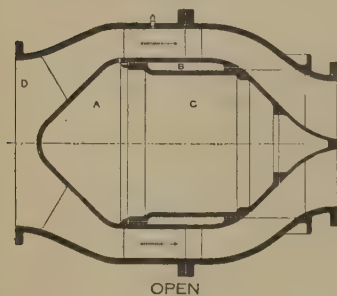


Fig. 30A.

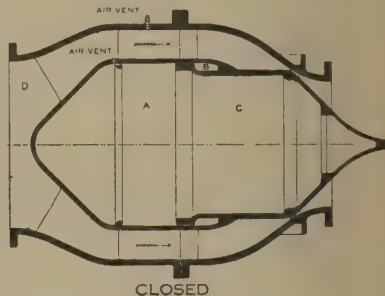


Fig. 30B.

long period of inaction, and it is in this respect that cheaply made goods fail and cause endless trouble and not a little expense.

For the control of water in large pipe-lines, there is a tendency to substitute plunger valves of the Johnson type for sluice or gate valves. They certainly possess several advantages.

The essential features of the Johnson plunger valve are shown in Figs. 30A and 30B. It will be observed that the valve is inserted in the pipe line and operated by direct pressure. The body of the valve is cylindrical, and, together with its casing, is finished to stream-line form so as to offer the minimum of obstruction and possibility for the formation of eddies. The valve consists of a cylinder, which in the closed position rests against a seating in the main, but when open is telescoped

back into its casing so as to leave a large free waterway. Fig. 30A shows the valve open, and Fig. 30B shows it closed. Referring to the lettering, A is the valve proper or plunger, while the casing or cylinder into which it is telescoped is shown at B, C being the outer casing which forms part of the pipe line. Casing B is held in position within A by ribs. Valve A is guided in its movement in two places. The front portion works in a stuffing box in B, while the rear of the valve, which is of larger diameter, is in the form of a packed piston moving in an enlarged cylindrical portion of B. The valve is operated by admitting pressure water into B behind A, at the same time putting space D into communication with the atmosphere. Under these conditions the valve will move to the closed position as indicated, while if the reverse conditions are in evidence the valve will open.

Johnson valves are built in quite a number of different types. There is, for instance, the Type C Valve. The Type C Valve is hand operated, but may be equipped for electrical operation by the addition of a motor control unit geared to the hand mechanism. The inlet and outlet diameters are usually, but not necessarily, the same.

It is a one-way valve, and is suitable for purposes where the flow is in only one direction. The rate of opening and closing is under absolute control, and the power required for operation is almost negligible.

This valve is particularly suited to conditions which require operation at partial openings, such as throttling valves in waterworks systems for reducing pressures in distributing lines and for regulating the flow in pipe lines. When operated electrically and equipped with pressure-controlled switches, it will automatically throttle the flow in a pipe line and hold the pressure on the discharge side constant. When used for throttling, it is entirely free from vibration.

The control of the Type C valve is located inside the valve. There is no external control valve or outside discharge during operation, as in the case of Types A and B. The absence of all external control valve and piping is especially advantageous in the handling of very high pressures.

Automatic Stop Valves.—In the main between the storage reservoir and the service reservoir, and on such other points on large diameter mains as may be deemed advisable, it is usual to install some form of valve which will automatically close in the event of a burst. The bursting of a large main in

120 PRESSURE, AND ITS EFFECT IN PIPES

a populous area is very troublesome, and may in some localities cause a dangerous flood. Despite the general improvement in pipe manufacture, mains do burst quite often, especially during hard frosts, and the provision of automatic stop valves at suitable points is a very wise and necessary precaution. There are two types in general use. The first and rather older

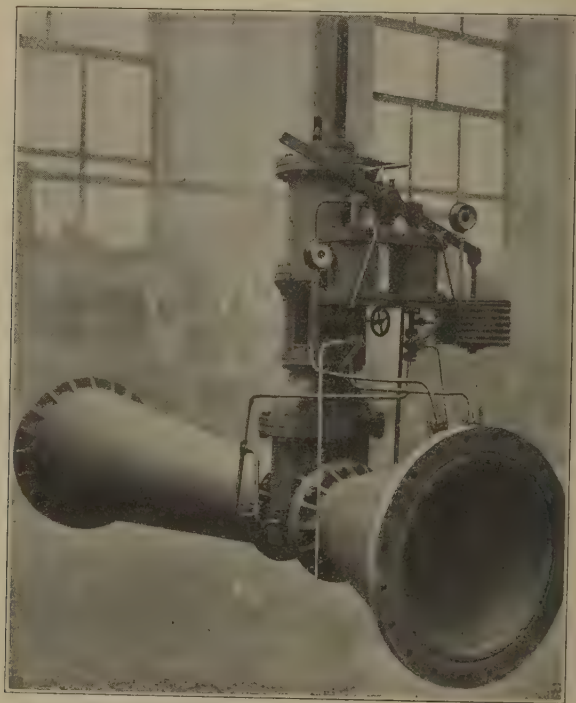


Fig. 31.

one consists of a balanced disc valve, which is held in the open position against normal velocities of flow. Velocities above a certain limit for which the valve is set, upset the balance of the mechanism, and the flow of water closes the disc. This, however, in accordance with hydraulic laws, is only permitted to take place slowly, the actual rate of closure being also adjustable.

The more modern method applies the ever-useful principle of the Venturi, as seen in the Edwards valve, illustrated in Fig. 31. It will be observed that the pipe line is much reduced in diameter at the point of insertion of the valve, this contraction being in the nature of a Venturi tube, the principle of which is taken into account in the operation of the valve, making possible the accurate adjustment necessary to ensure closure at any desired rate of flow. The valve, a sluice valve, is operated by the piston in the large cylinder at the top, and is normally at the top thereof, while in one of the small cylinders seen in front of the main one is another piston normally at the bottom of its stroke. The space above this piston is in direct communication with the smallest diameter of the Venturi tube, while the space below communicates with the upstream side of the tube, as seen in the front of the view. Thus there is a difference of pressure on the two sides of the subsidiary piston, but the weight shown keeps it at the bottom of its stroke. Should, however, an abnormal flow occur the pressure balance is upset, the weight falls, and by reversing a four-way cock puts pressure on the top of the main piston, and the valve starts to close.

The motion, however, in accordance with hydraulic laws, must take place slowly, and this is arranged for by a pin on the tail rod, which, it will be noted, also forms an indicator giving the position of the valve, slowly closing the valve on the water-pressure admission pipe as it travels downwards. The valve is re-set by first equalising the pressure on both sides by means of the bye-pass valve, after which the lever is pulled over to its original position, which, admitting pressure to the underside of the operating piston, opens the valve, the subsidiary piston in the meantime being forced to the bottom of its stroke and held there by the weight on the end of the lever until conditions of flow call for another action.

By altering the weights in accordance with a carefully calculated scale, the valve can be set to operate at any rate of flow, while in the case of a burst on the downstream side of the valve, a special arrangement comes into action. This takes the form of a pitot tube fitted to face the flow in the contracted area, and connected through the four-way cock with the upper side of the operating cylinder. The dynamic force due to the velocity of the water induces a pitot head of sufficient magnitude to start the motion of the operating piston and to cause it to close. As it does close the static

pressure on the main begins to rise and the piston speed is accelerated.

Distribution.—Of the numerous items of equipment which enter into the street to street and house to house distribution

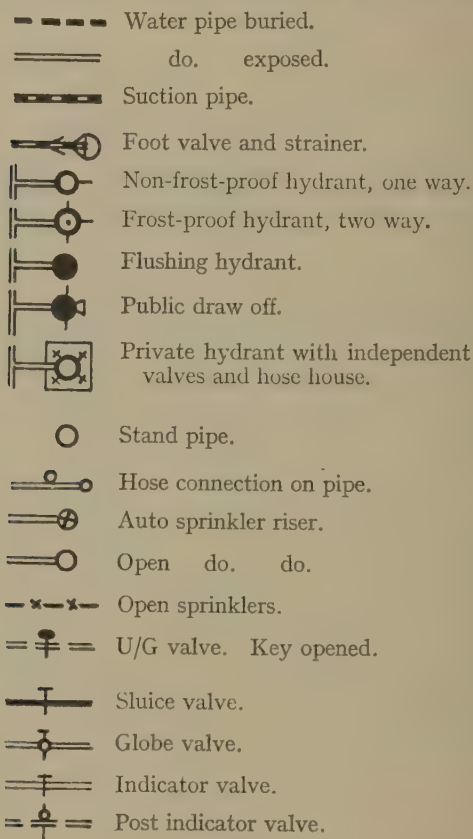


Fig. 32.

of water it is not necessary to treat at any length, as such equipment is fully listed in makers' catalogues. One essential is a good map showing the position of each main, branch, valve, hydrant, connection, etc., very plainly, together with the depth of the pipes and their position relative to stated

points on the surface. Nothing is more troublesome than to have to hunt about to find hidden pipes, etc., which have probably been forgotten for a number of years. If a standardised system of conventional signs are used a considerable amount of information may be recorded on a 5-ft. ordnance map, and the diagram in Fig. 32 is suggested as covering most requirements in this direction.

Meters.—Apart from the more usual equipment of a waterworks distribution system there is the matter of meters. In this country it is only large premises, such as hotels, blocks of offices, etc., which with most trade users are on metered supplies, although in America it is almost universal. Between the ordinary run of meters there is not much to choose; some are positive meters which depend upon the displacement of a given volume of water by a piston or disc, while others are inferential meters, the essential feature of which is a small turbine designed to pass a given volume per revolution. Both are accurate within commercial limits, but the latter class is perhaps in more general use than the former. In this country meters are not subject to much damage by frost, but in northern latitudes it is usual to provide the meter with a relatively weak base of cast iron—a frost bottom—which will fracture before other parts of the machine get damaged. Replacement is a simple matter.

For the measurement of large flows of water there is but one satisfactory instrument, the Venturi. While much could be written upon this interesting hydraulic instrument and the principles involved, it is now so well known by waterworks men and so ably described by the makers, Messrs. Kents, of Luton, that brief mention will suffice here. It is a relatively costly piece of apparatus, but it is essential for the recording of water sold or purchased in bulk, of the output from pumps, and of the amount handled by filtration and purification plants in general.

Mention might also be made in passing of the so-called waste-detecting meter. Introduced many years ago by Mr. G. F. Deacon, formerly waterworks engineer to the Liverpool Corporation, this meter records upon a chart the *rate* of flow at any instant. Fixed on the mains at suitable points, these meters have a certain value in enabling inspectors to check up night flow. If a really intelligent use is made of them, small leaks and bursts can be tracked down pretty rapidly. The reason why the system is sometimes regarded

as of little value lies in a perfunctory in place of a constant and intensive following up of the leaks indicated by the meters. Waste detection pays, and pays well, if it is a continuous business in the hands of an inspector told off for the work ; and especially is this the case if water is pumped or purchased in bulk.

The Deacon meter is admittedly rather costly to install, especially if many are required on a system, and a recent introduction is the Pitometer. This, to all intents and purposes, is a portable meter which can be inserted in the mains in a very simple manner through a small ferrule. One instrument, therefore, can be made to serve a large area.

CHAPTER XV

PUMPING WATER

Steam Pumps.—Upon the subject of waterworks pumping a vast amount might be written. The original matter presented by Mr. Slagg has been entirely deleted as it treated in the main with non-rotative steam engines of a type now quite obsolete. Conditions during the last decade have practically revolutionised pumping practice. The cost of coal has put small steam pumps right out of court, and large steam pumps, while economical in operation, are so costly to build nowadays that they are only installed in very rare instances. Non-rotative engines of the Cornish class have practically disappeared, in some instances engines of the Davey type are doing good service, while the Worthington engine is a compact unit, reasonable in first cost and fairly economical of steam. But so relentless has been the call for economy, that when steam is finally selected as the motive power, the selection of a unit usually lies in either a triple-expansion rotative engine of the class shown in Fig. 33, or Unaflo engines when centrifugal pumps are decided upon for low lifts. In the former case the pumps are usually placed below the engine and operated direct from the crosshead, and a combination of lift pumps for the wells and surface pumps for handling the water so lifted is quite a common arrangement which works well.

The force pumps are usually of the simple outside packed plunger type with the same stroke as that of the engine. The well-pumps would be of a type best suited to the special conditions, and when it comes to pumps of relatively large capacity located in small boreholes, at perhaps considerable depth, special considerations enter into the design. In the first place, a diameter limited perhaps to 24 inches or less renders it imperative to make provision for drawing up the whole pump and valve assembly to the surface. The pump would obviously be drawn up by the rods, these being made with coned joints coupled with cotters, while the suction

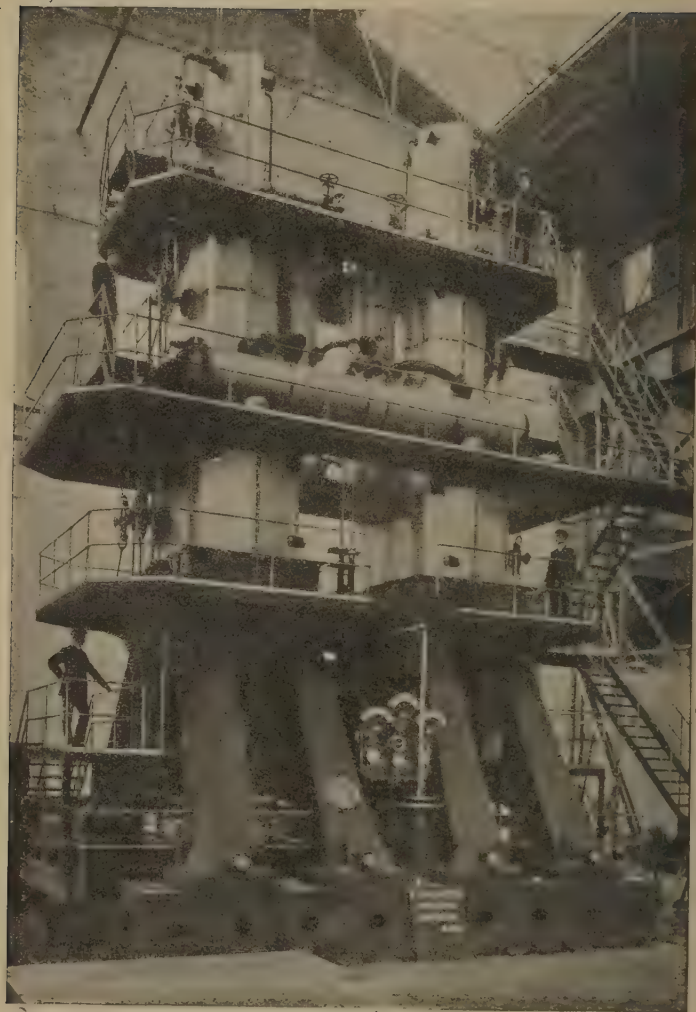


Fig. 33.

valve assembly terminates in a bulb-ended stem which is gripped by the usual form of "fishing" tackle. Numerous designs for this class of pump have been put forward from time to time and the arrangement shown in Fig. 34 depicts the most recent practice. The line terminates in a cast-iron suction pipe with the necessary slots, above which comes the clack-box or suction-valve assembly and then the working barrel, these together with the rising pipe being assumed to be flanged. Within this barrel works the bucket constructed as shown on the left of the view.

The bucket *a* attached to the pump shaft *b* is provided with a gutta-percha ring *c*, the holes *d* permitting of water gaining access behind the ring and so expanding it against the walls of the pump barrel at *e*. Two rows of valves are shown in the diagram, the usual number being about six, and they decrease in diameter towards the top. The whole therefore takes a conical shape, and the actual valve area can be varied by the rate

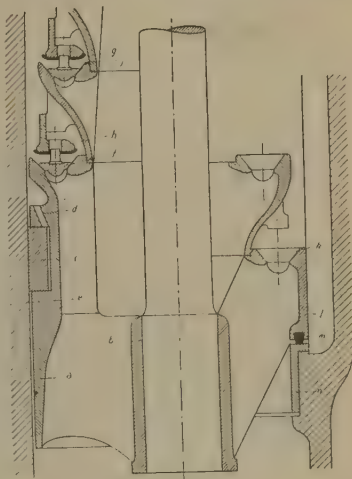


Fig. 34.

of decrease and by the number of valves fitted. Usually the clack has one extra valve to ensure easy flow. The multiple valves *f* are of the "Pernis" type and are of gun-metal, the disc *g* being of leather, and above which there is the guide-ring *h*. The lugs *j* on each valve project through the disc into the guide-ring. The load is taken on the valve-ring, which fits metal to metal with its corresponding seat, while the leather disc pressing down on the valve-ring makes a tight seal. The suction-valve assembly is built up on similar lines as depicted in the right-hand view, which shows the lowermost valve seat at *k*. The bottom portion *l* is fitted with the gutta-percha ring *m* dovetailed into it, and when lowered down this seat makes contact with the G.M. ring *n*. It will easily be seen that this arrangement ensures straight and easy flow of water, together with a large valve

area and small lift, all of which make for durability, which is so essential in a deep-well pump.

The judicious use of the Unaflow engine is seen in the recent installation of low lift centrifugals at the Littleton pumping station of the Metropolitan Water Board, gas, oil, and electric power having been given very careful consideration before the final choice of steam was made.

The duty of the waterworks engineer in connection with installing a pumping plant is to obtain a plant reasonable in first cost which will operate at a definite cost per *water* horsepower. This is the "acid test" of any pumping plant, and

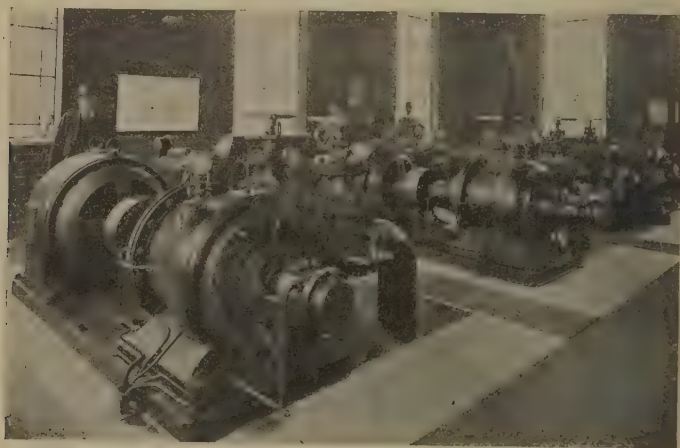


Fig. 35.

it is upon this guaranteed figure that final choice largely depends. Under present-day conditions it is only large, modern, and costly steam plants which can compete with those operated by electricity or internal-combustion engines.

Electric Power for Pumping.—With the extended distribution of electrical energy on the one hand, and the improvement in centrifugal pumps for both high and low lifts on the other, and the compactness, reliability and low cost of the installation as a whole, the electrically operated pumping plant is being more and more favoured. In fact, if electric power at reasonable cost is available within any reasonable distance at all, there is little inducement to install any other

form of power. True, both gas and oil engines can often show cheaper operating costs, but the difference is not always sufficiently pronounced to make this alternative attractive. The modern electric motor is so compact, so efficient, and requires so little attention, that when electrical energy is available there should be very good reason for turning it down. Costly electric power to-day is inexcusable, and is usually the result of bad management at the hands of the power concern. At the same time, the latter are up against their own special difficulties, and arrangements to take power at certain periods of the day may result in its being obtainable at a very favourable rate. The value of such an arrangement should not be lost sight of. The illustration in Fig. 35 of a pumping station for Dudley, operated by the South Staffordshire Waterworks Company, shows how compact such an installation may be.

It is nearly always possible to handle water economically by means of a centrifugal pump. It is now practically a standardised production, and the multi-stage pump is capable of meeting all requirements of head. Even deep well pumps are now built on this principle, and this in fact is a recent and very interesting development of the centrifugal pump. Briefly, the method involved is to construct the pump with a very small overall diameter and suspend it from the rising pipe. The latter is formed with accurately machined and spigotted flanges, which ensure its alignment, and within it the shaft, built in lengths and connected by similarly machined couplings, is supported by steady bearings. The whole equipment can be lowered several hundred feet when necessary in a bore of but 12 in. or 15 in., the arrangement of the motor at the well head being shown in Fig. 36. An alternative

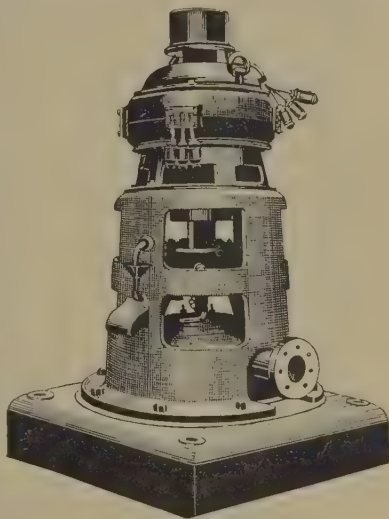


Fig. 36.

arrangement, and one which has been widely adopted, is to drive the pump by internal combustion engine, a gear box of the high efficiency oil-bath type taking the place of the motor shown at the well head.

Internal Combustion Engines for Pumping.—One of the outstanding features of modern pumping practice, both in this country and America, has been the widespread adoption of the internal combustion engine. Of course, for a number of years the gas-engine-producer combination with a treble-ram pump or well pumps of the lift type, as the case may be, has proved a reliable and economical arrangement, and one which for powers up to 100 horse-power or so may still be regarded as modern; but the high cost of fuel, and especially of the anthracite fuel as generally required by producers, has gradually caused it to give way to the oil engine.

The plant at Rugby, as shown in Fig. 37, is a typical instance of modern pumping practice, showing a four-cylinder 200 horse-power Diesel engine driving a centrifugal pump through gearing. This again is regarded as good modern practice, in that it enables both engine and pump to run at economical speed, that for the engine being round about 250 r.p.m., and that for the pump 1,000 r.p.m. more or less according to conditions. So rapid has been the evolution of the oil engine of late years, that engineers not actually engaged upon their manufacture are apt to be somewhat misinformed just as to what constitutes the modern oil engine in its various forms.

The following is an extract from an article by the writer in *Water Works Engineering*, New York:—

“The modern oil engine has proved itself a remarkably efficient prime mover, and its extensive application to pumping is a development of considerable interest to waterworks men. While for handling large volumes of water under conditions calling for continuous operation the steam driven waterworks engine still holds the field for economy and reliability in operation, the small steam plant is virtually defunct, its place having been taken on the one hand by electrically driven units in locations where electric power is cheap, and by the oil engine driven pump in others.

“The term ‘oil engine,’ however, is a very vague one, and may mean anything from a small high-speed engine of the automobile class designed to run on kerosene, to a 1,000 horse-power Diesel capable of operation on the lowest grades

of fuel oil and even tar and other liquid combustibles. The first is mainly suited to small standby plants, while the latter would figure in the operation of a large low lift centrifugal,

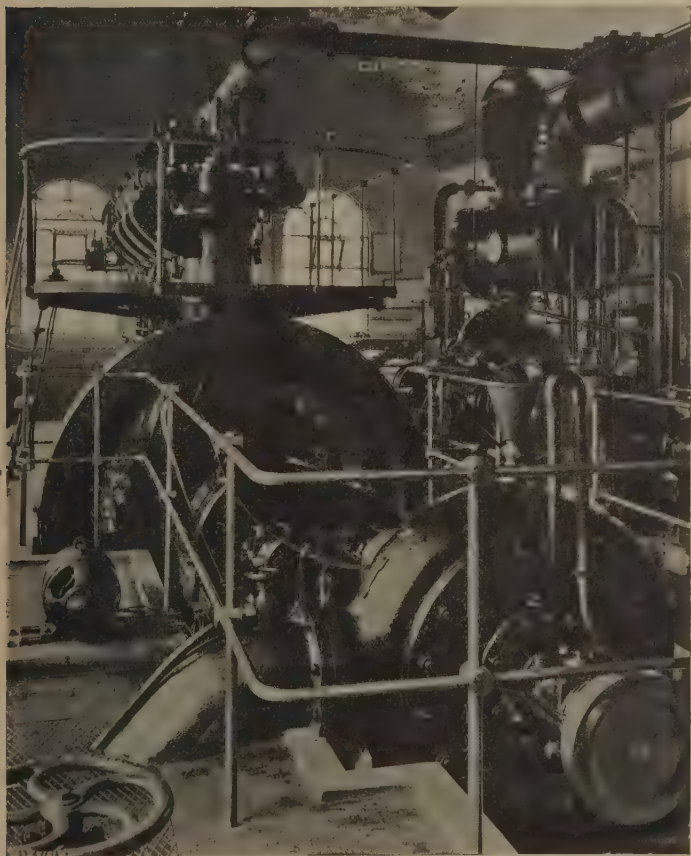


Fig. 37.

and its application in waterworks practice is therefore somewhat limited. The modern oil engine is intended to function on low-grade oils which are naturally the cheapest, and it is without exception an automatic ignition engine; that is to

say, the injected fuel is ignited by the heat in the combustion chamber as opposed to the spark plug in the gas or gasoline engine, kerosene engines being virtually gas engines operating on gasified fuels. These latter engines, moreover, operate on the Otto cycle in which the charge of combustible is inducted into the cylinder and compressed. In the oil engine air alone is inducted and compressed and the fuel injected into the compressed air.

“This principle of automatic ignition which is the basis of all oil engines is nothing new, and its credit stands to Stuart Akroyd, an Englishman, but it remained with Diesel to introduce such engines on a really commercial scale. Hence the term ‘Diesel engine.’ This term, however, is loosely applied, and in addition oil engines are freely referred to as heavy oil engines, solid injection engines, cold starting engines, crude oil engines, high compression engines, hot-bulb engines, and semi-Diesel engines, and with little or no attempt at classification. Outside of those directly concerned with the design and construction of oil engines, therefore, considerable ambiguity exists as to what is meant by these terms. There are really three distinct types: the Diesel with air blast injection; the cold starting engine with solid injection; and the hot-bulb engine, also with solid injection.

“**The Diesel.**—The true Diesel engine operates on the cycle adopted by Diesel. The essential feature is the compression of the air to about 400 lb. and the injection of the fuel by a high pressure air blast which serves to atomise it. The compression pressure is the maximum pressure. The heat of the compressed air charge ignites the oil, which burns without explosion, the gases doing further useful work during their expansion. The Diesel engine, by reason of its peculiar cycle and its air blast injection, is able to show the highest fuel economy of all oil engines, and, built as either a two-stroke or a four-stroke engine, can operate when required upon almost any liquid or semi-liquid combustible.

“**Cold Starting Engines.**—For moderate powers the Diesel has never appealed to engineers to any extent, on account of its being somewhat delicate and complicated by the high pressure air compressor. The idea of injecting the fluid by mechanical means—solid injection—is not a new one, and the idea of making up for the comparatively poor atomization obtained in the early days of oil engines by maintaining a hot spot for ignition, is seen in the hot-bulb engine. The

high compression engine is the result of attempts to build an engine without a hot bulb and without air blast injection. The high compression is to enable sufficient heat to be maintained in the combustion chamber to effect ignition, even when starting from cold, hence the term 'cold starting engine.' But the term 'high compression' is again a misnomer because the actual pressure adopted is that decided by the builder. The success of these engines has been largely due to the perfection of the mechanical atomiser. In general construction, beyond the elimination of an integral compressor the engine resembles the Diesel. In the matter of fuel consumption and in its ability to run on low grade fuels it compares very favourably with the Diesel, and being more robust and simpler is generally a more attractive proposition up to, say, 400 horse-power. The cycle of operations, however, is different to that of the Diesel. The injection of the oil causes a rise of pressure due to an initial explosion on injection, so that the maximum cylinder pressure is greater than the compression pressure and may be considerably so. This is of no great moment in the present connection, beyond the fact that great strength in certain essential details is necessary and the engine is by no means cheap to build.

"Solid Injection Engines with Low Compression.—This class of engine, with the exception of one or two hybrid designs, is generally built as a two-stroke. For some obscure reason they are termed semi-Diesel engines, notwithstanding the fact that they have nothing whatever in common with the Diesel and differ materially from both it and the cold starting engine. In the first place, they operate mainly on the two-stroke cycle and mostly with crankcase compression. In consequence they are simple to construct and comparatively cheap in first cost. The design, in most cases being unsuited to sufficiently high compression to commence or maintain ignition, incorporates a dome-shaped appendage to the cylinder head which is not water cooled and may be externally heated before starting. Hence the term 'hot-bulb engine,' which so far as most of these engines is concerned is a true cognomen. The cheap first cost is attractive and so is the simplicity, but the fuel consumption does not compare favourably with other types, and the crankcase compression entails a heavy consumption of lubricating oil and has other minor shortcomings.

"Realising that simplicity and cheapness in first cost are obtained at the expense of enhanced operating costs, designers

have recently evolved improved types of these semi-Diesel engines. The first takes the form of substituting pump induction for the crankcase compression. The underside of the piston is enclosed and constitutes the pump, which gives better scavenging and induction and eliminates the shortcomings of crankcase compression. Another improvement is the raising of the compression pressure. The hot bulb is therefore practically eliminated, its place being taken by either some form of electric starter or an externally heated ignition plug. After starting sufficient heat is maintained to ensure ignition, and the engine becomes a surface ignition engine rather than a hot-bulb engine.

“Types compared from Users’ Point of View.—All these engines are entirely suited to pumping duties, whether ram pumps or centrifugal pumps are decided upon. In the latter case most large low-lift pumps can be direct coupled to the engine. Gearing is, of course, required in the case of reciprocating pumps, but with the introduction of high efficiency gearing enclosed in oil bath cases gearing troubles have largely been overcome, even to the extent of warranting a gear set between engines and centrifugal pumps to ensure an economical speed for both. A further successful application is the operation by oil engine of centrifugal borehole pumps, a method of borehole pumping which is making rapid strides. The Diesel engine, with its comparatively high cost and need for skilled attention, is principally attractive in powers of over 500 horsepower where continuous operation is the rule. Its superior fuel economy and its ability to work upon very cheap fuels pays under such conditions. For the general run of pumping duties the cold-starting solid injection engine is likely to prove the best proposition. The fuel economy is high and the engine has all the advantages of the Diesel without its complications. It is somewhat less costly to install and can be started from cold without any preliminary preparations. In British waterworks practice it has proved highly successful. The hot-bulb engine is essentially a unit for the operation of small plants and especially those intermittently operated. In such cases a high fuel economy does not show up very strongly, while the reduction of capital sunk in such plants is very important.

“Compactness and comparatively light foundations are again favourable to this type, which has a distinct sphere of usefulness which the improved types referred to tend to extend.”

The oil engine-driven borehole pump mentioned above has made rapid strides of recent years, and mention in particular might be made of the work of Messrs. Ruston Hornsby, Ltd., at both the Bilston and the Broadstairs waterworks. Fig. 37A indicates the usual lay-out of such a plant in duplicate, with vertical three-cylinder engines operating borehole pumps through the medium of clutches and high efficiency bevel gearing. In the gear case is also housed the necessary thrust bearing, an oil-circulating system, and a coil for cooling the oil. An overall efficiency of over 70 per cent. is obtainable with a well-conceived plant of this description with a fuel consumption of about 0.65 lb. per pump horse-power hour.

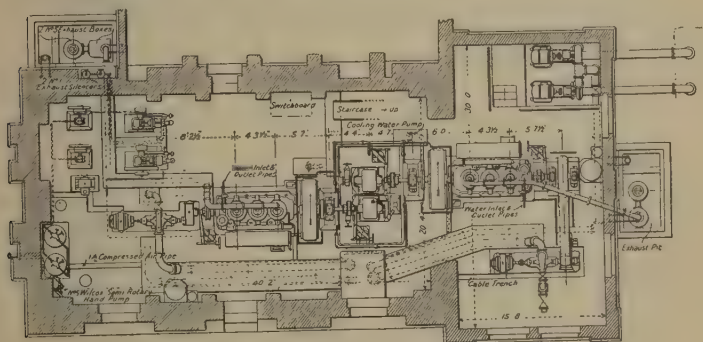


Fig. 37A.—Rumfield's Engine House.

Air Lifts.—In connection with borehole pumps a somewhat limited application has been made of the air lift in this country, although it is pretty common in America. It has one great outstanding advantage, in that there are no moving parts at great depths, the sunken portion of the apparatus being nothing more than an air injection nozzle of special form, termed the "footpiece." The air acts upon the water by a peculiar process of aeration, which causes it to travel up the rising or "eduction" pipe to any reasonable height as may be required. This pipe usually terminates in a mushroom-shaped hood. There is no doubt as to the effectiveness of the air lift as a means of raising water from boreholes, but despite somewhat extravagant claims to the contrary, it still remains wasteful in operation. This is partly due to the method on which it functions, partly in many cases to badly designed footpieces, and finally to the fact that most small air compressors are themselves inefficient machines. The result is

that an air lift operates usually at an overall efficiency of about 30 per cent. Nevertheless, for small boreholes the virtues of the air lift must not be overlooked because of its inefficiency; for many situations it is a really useful piece of apparatus. But with the perfection of the aforementioned centrifugal borehole pump it is doubtful if much progress will be made with air lifts. They cannot compete in cost of operation with the mechanical pump, and are not usually less costly to install.

Hydraulic Pumping.—In connection with small works there are two means of pumping water which often prove useful. The first is the hydraulic ram. This simple piece of apparatus may not be "efficient" so far as pumps go, but it is very well suited to pumping part of a stream of running water to an elevated tank. The operating cost is nil, and unlike the hydraulic turbine, the cost of installation, apart from that of the ram itself, is very small.

The object of the apparatus is to pump water to a considerable height by utilising the potential energy of a supply of water placed at a smaller height. At the joint A connection is made to a length of pipe, usually 10 ft.

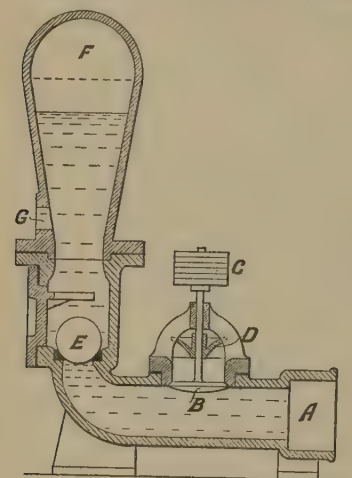


Fig. 38.

to 20 ft., leading to the supply of water to be utilised. Connection is made at G to the receiving tank to which the water is to be pumped, so that the air contained in the bell F is compressed to a pressure corresponding to the head of water connected to G. When the valve B is shut the water in the pipe A is stationary. The weights C applied to the valve B are sufficient to overcome the pressure in the pipe A and thus cause the opening of the valve. The water now begins to acquire a velocity and escape through the valve B, thus converting the whole or part of its potential energy into kinetic energy. As the water escapes through the valve B it meets the guide D and is deflected, causing an upward pressure on the valve spindle sufficient to overcome the weight C and close

the valve. The water in the pipe A, having a velocity and corresponding kinetic energy, is now entrapped in the pipe, and as this energy cannot be dissipated and cannot continue wholly in its present form, as the velocity of the water has been checked, it is evident there must be a conversion of energy to the pressure form.

This conversion causes a heavy pressure to be generated in the pipe A, and when this pressure has risen above the pressure in the chamber F the ball valve E will be raised, and water will flow from A to F as long as the pressure is maintained in the pipe A greater than the pressure in F. During the entry of the water from A to F the pressure in F is increased owing to the compression of the air. This increase of pressure overcomes the pressure acting at G, and there is a consequent flow through G to the elevated tank. On the closing of the valve E the pressure in A again returns to that due to the smaller head, and the valve B is free to be operated by the weights C causing a repetition of the operation. Thus potential energy is converted into kinetic, kinetic to pressure, and pressure to potential energy.

While the foregoing depicts the essential principles of the hydraulic ram, it does not necessarily show the best form. Modern rams have usually mitre valves in place of ball valves, and the proper weighting of these has a good deal to do with the efficiency of the ram in service. Another point to bear in mind, too, is that there is only one concern in this country, Blakes of Accrington, who make a speciality of rams, and cheap articles which are merely a rough ironfounder's job usually end in giving trouble. While essentially simple, this piece of apparatus calls for scientific design and construction on engineering principles.

The second method is the use of a small hydraulic turbine. Water power has already been touched upon, but it remains here to direct attention to the fact that in the case of river supplies, the pumping necessary can often be very conveniently effected by this means.

It is not quite such a simple proposition as the hydraulic ram because of the incidental works necessary to the installation of any turbine, and a fuel-operated plant may often be a better proposition; but the possibilities of water power should certainly be investigated, bearing in mind that an electrically driven pump may often be operated by current generated by water power perhaps a mile or two away, although the direct driven plant, when feasible, is obviously less costly.

CHAPTER XVI

PURIFICATION

APART from the fact that the public demand pure water and the law stipulates that they shall have it, modern methods of water purification are of the utmost interest to the waterworks engineer.

During the middle of last century, when public supplies were being put into operation all over the world, the water had to be pure in its raw state because methods of treating it were confined to slow sand filtration. By means of improved methods it is now possible to render, although perhaps at considerable cost, comparatively dirty water, not only clear, bright, and sparkling, but bacteriologically pure. The latter is very important, and it might be said that the first step in the consideration of any source of supply would be the examination thereof by a qualified chemist. It is the chemist, too, who would decide to a great extent the methods to be adopted in rendering any supply absolutely safe, although putting these methods into practice is engineering work pure and simple. On the larger and older works it is usual to find somewhat large areas laid out as sand beds. Sand beds are effective in removing suspended organic matter although the process is slow. In consequence the area to be laid out is large. The sand bed also functions as a purifier in this respect, that a gelatinous film settles on the top of the bed, and it has the property of eliminating a large number of disease germs, such as *B. Coli*, if present. There are many engineers who still hold that the sand bed is superior to any form of rapid filter, but be this as it may, the fact remains that they are costly to build and to maintain, and such troubles as peaty discoloration are not usually amenable to treatment in this way.

Hence it is that the majority of new works involving filtration plant make use of the *Rapid Filter*. Rapid filters are sometimes spoken of as mechanical filters, but the term is

a misnomer. Mechanical means may, as will be shown, be used for cleansing the bed, and this periodic cleansing is one of the essential features of the rapid filter, but the actual purification is on the same lines as the sand bed. It is all a matter of passing a greatly increased amount of water per unit area of filter.

This is made possible, first of all, by the injection into the water of a precipitant such as aluminic ferric, which coagulates

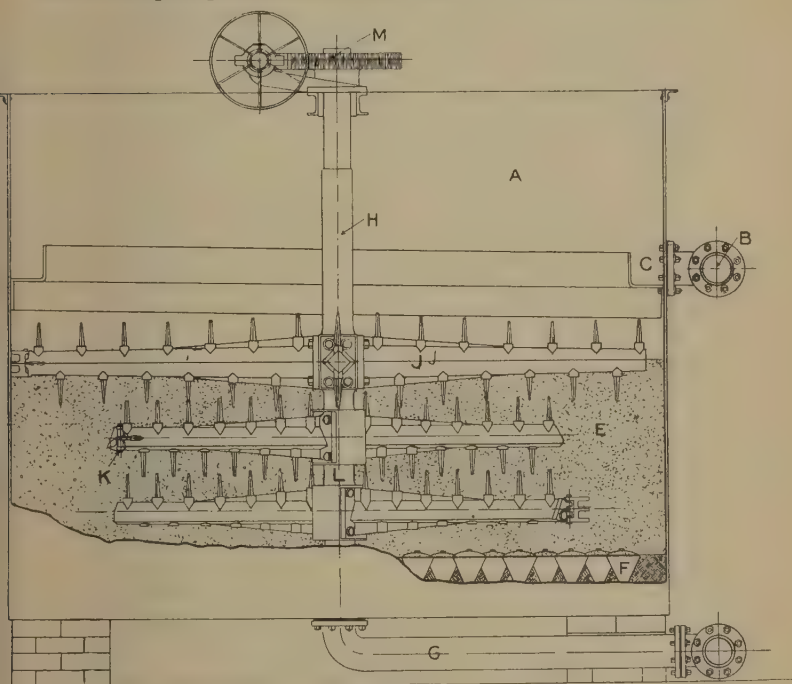


Fig. 39.

the suspended matter and renders it much easier of removal by the filtering medium. Secondly, means are provided for cleansing the bed at frequent intervals, two or three times per day or more if required, and in some designs the cleansing is continuous. The bed, therefore, can be worked at a very high rate, hence the term "rapid" filter.

Construction of Filters.—In Fig. 39 is shown a section of a typical rapid filter with rake cleaning gear, as made by Bells.

After the water has had the necessary chemical added to it it passes into the filter A through pipe B to trough C. After passing through the filter bed E it issues from nozzles F into clean water-pipe G. The washing of the filter is effected as follows: The washout valves are opened, and the inlet valves are shut. Clean water is passed through the bottom pipe and strainers F upwards through the bed putting the same into suspension, this water passing out through a washout pipe. This has the effect of putting the bed in suspension, at which time the wash valves are opened and water passes into hollow shaft H, hollow arms J, and through back-pressure valves K. Shaft L is then revolved by wheels M and carries with it in revolution the rakes. This process breaks up and stirs the bed, loosening the dirt, which is carried away by the wash water.

This system is a great improvement upon the old plain rake arrangement and is quite as effective, if not more so, than the propeller gear, the former being now confined for the most part to quite small units and some of the cheaper types of open filter.

The propeller gear, which was a feature of Mather and Platt's filters, has been discontinued in favour of the arrangement shown in Fig. 39.

Rapid Filters having a Continuous Washing Arrangement.

—One of the objections sometimes raised against the rapid filter is the fact that it incorporates a mechanical washing gear which has to be periodically put into operation, necessitating stoppage. What is termed the drifting sand filter has recently been introduced in an improved form with the object of overcoming the difficulty. Chemical coagulants are used as they are, with other rapid filters, and a special form of apparatus for use in conjunction with the type under discussion will be dealt with presently. The main difference between the continuous washing or drifting sand filter and those with mechanical wash gear is, that while the latter usually require washing at least every day the former will operate continuously for from three to thirty days, according to the state of the raw water. Hence a plant of this kind can be looked upon as largely automatic. The general arrangement of the filter is shown in Fig. 40.

After being chemically treated the water enters the filter, partly by a standpipe *a* and partly through bye-pass *b*. The sand washer is shown at *c*. Within this sand washer the raw

water-pipe *a* is constructed on the Venturi principle, and fresh sand therefrom is induced into *a* and passes up with the

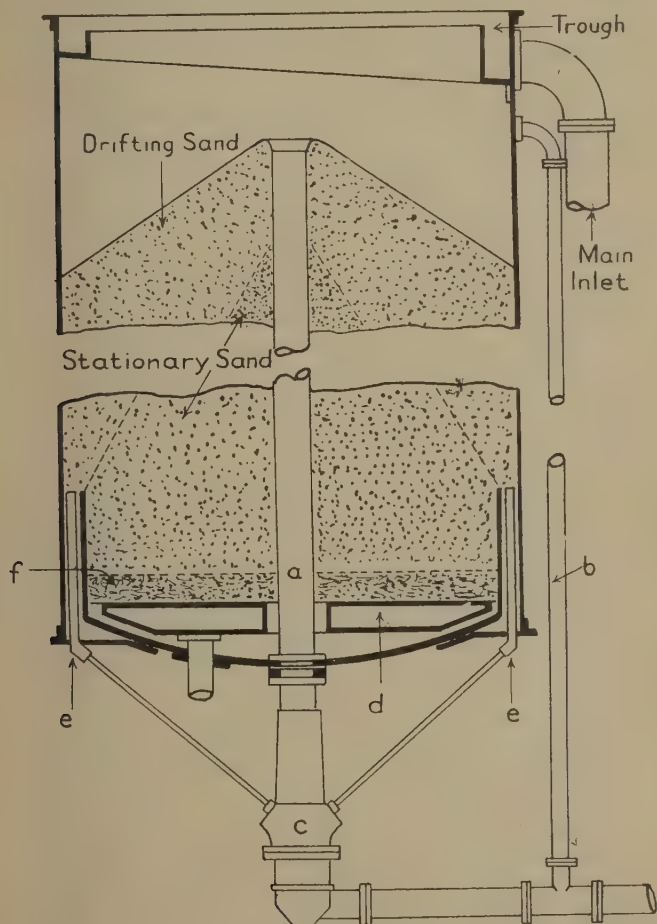


Fig. 40.

water over the top of *a*, where it settles on the bed in a sugar-loaf formation. The sand thus deposited is continually drifting away. In the meantime the water is passing through

the sand and being filtered, eventually passing away through pipes *d*. Simultaneously the sand-washing process is going on all the time, the sand being taken away to be cleaned through the extractors *e*. A portion of the sand bed is stationary, but part of it, as shown, is in the form of a hollow cone of continuously drifting sand, hence the name given to the filter. This drifting sand is being continuously cleaned in the sand washer from which the dirt is expelled. The stationary body of sand is fairly hard and compact, but the drifting sand is buoyant and spongy. Occasionally the latter is washed by a reverse flow of the water, a process common to other types of rapid filter. The total depth of sand is about 9 ft. and it rests on a 10-in. bed of a gravel *f*. The sand washer *c* is of special design, but really resolves itself into a series of jets which impinge on the sand as it falls into the washer, to the bottom of which it gravitates, and is then drawn into the pipe *a*, as before stated, by Venturi action.

The bye-pass arrangement permits of the rate at which the sand is moved being under control, and the rate thereof in turn depends upon the state of the raw water—the dirtier the water the more washing required by the sand, and hence the less water bye-passed via *b*. The system is applicable to both closed and open filters, while it may be added that in a large filter several vertical pipes, with their attendant sand-drifting pipes and washing gears, may be installed. Sharp quartzite sand is the filtering medium used. The rate of filtration is about 100 gallons per square foot of filtering surface per hour with the open type, and about 40 per cent. more with the closed type. The operating head necessary is about 30 ft.

It is not necessary that any special form of coagulating gear should be installed, but that adopted by the makers of this filter is worthy of a brief description. It is shown in Fig. 41. The essential parts of the apparatus are the three cylinders *a*, *b*, and *c*, controlled by the charging cock *d* and the orifice outlet cock *e*. The water to form the standard solution is drawn from main pipe *f*, the solution entering the pipe at *g*. *g* itself is a mixer in the form of a cylinder closed at both ends and contains a series of tubes through which the water passes, and during its passage it takes up coagulant through small orifices in these pipes. Blocks of chemical are put into the dissolving tank *h*, which is a perforated cylinder inside *a*. The liquid from *a* passes into *b* and escapes therefrom through

cock *d* at the open-ended branch. The solution which issues from *a* is a strong solution.

In cylinder *c* is a hydrometer which controls the strength of the solution, the strong liquor entering at the top and water at the bottom, the outlet being at midheight.* The hydrometer in *c* has needle valves top and bottom by which the

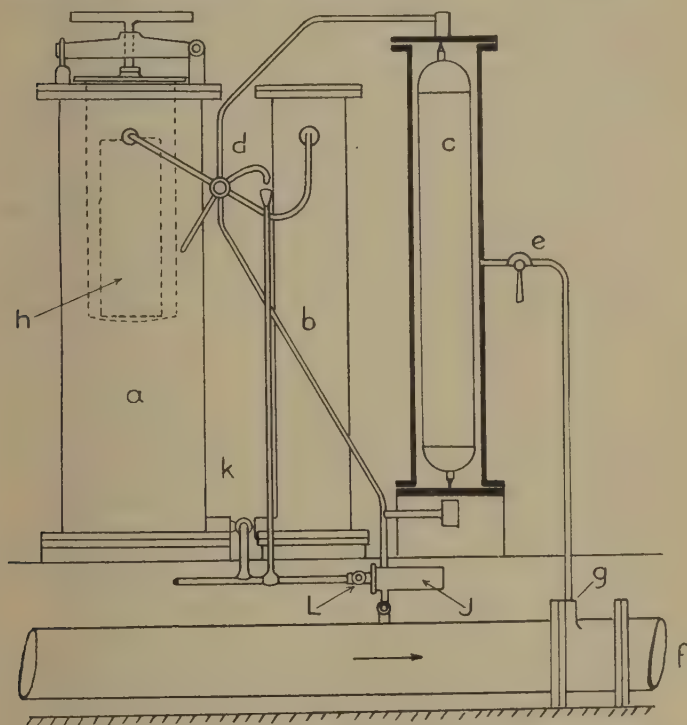


Fig. 41.

water and solution valves are under control, and a little thought will make it clear that the arrangement can be adjusted to keep a solution of standard strength in *c*.

By the entry of the strong solution at the top and the water which is lighter at the bottom, a proper mixing thereof will result. The outlet valve *e* of cylinder *c* has a series of definite adjustments to which its opening may be set in accordance

with the quantity of water requiring treatment which is passing. When cock *d* is in the working position water from pipe *f* will flow to the underside of *c* or to *d*. In the latter case it enters top of *b* out of the bottom thereof to *a*, where it picks up coagulant, thence through *d*, entering cylinder *c* as a strong solution. After being weakened to standard strength it passes out through *e* to *g*. If the cock *e* is turned to the charging position standard solution can for a time still be drawn from *c*, and during this time a fresh block of chemical can be put into *a* through the door on the top thereof.

Operation.—Rapid filters perform their natural functions without attention; the chemical plant when once adjusted requires little more beyond a periodical filling up. The washing is quite a simple matter and resolves itself into closing certain valves and opening others, or, as just described, it may be continuous. The normal time for cleaning is about ten minutes; it is done at periods according to the duty demanded of the filter and the state of the raw water. This must be ascertained from experience in each case. Makers of rapid filters usually give a guarantee of capacity under specific circumstances, which vary, of course, according to conditions. It is a great mistake to attempt to force these filters; an ample size should be installed; several may be necessary for a large plant. The average rate of working is 100 gallons per hour per square foot of filtering medium. The area of the latter is the controlling factor; the depth has nothing to do with it. In fact, when the filter is in use and the bed is "ripe" the top inch or so is mainly effective for cleaning the water. There is no doubt they constitute an ideal means of coping with large or small quantities of water, while at the same time occupying little space and costing little for maintenance; not only will they render the filthiest water perfectly bright and clear, but they have been also proved capable of removing typhoid and other disease germs, including the elusive cholera bacillus.

It is, however, advisable to chlorinate the water when required for drinking, if there is much doubt as to its bacteriological purity.

Operating Results.—Generally speaking, rapid filters have given highly satisfactory results under a wide variety of conditions.

The Borough of Shrewsbury, with a plant consisting of twelve pressure filters with a total capacity of $2\frac{1}{2}$ million

gallons per day, use raw water pumped direct from the river through the filters to a water tower. The following figures relate to the results obtained, column A referring to the raw water and B to the filtered.

	A		B	
Colour and appearance in 2-ft. tube.	Brown, yellow, and turbid.	Opacity lines obscured at 10".	Clear and bright.	Clearness= $\frac{11}{10}$.
Presence of aluminium.	..	—	None.	
Total solids dried at 10° C.	..	11.4 grains per gal.	10 grains per gal.	
Alkalinity	..	3.5 grains cal. carb. per gal.	2.8.	
Hardness—				
Temporary	..	2.8°.	1.6°.	
Permanent	..	2.2°.	3.4°.	
Total	..	5.0°.	5.0°.	
Parts per 100,000.				
Free or saline ammonia ..	..	0.0014.	0.0018.	
Albuminoid ammonia ..	..	0.1.	0.008.	
Nitrogen as nitrates ..	..	0.065.	0.063.	
Oxygen absorbed from permanganate in 3 hrs. at 80° F.	..	0.19.	0.012.	
Chlorine	..	1.8.	1.8.	
Metals	..	Trace of iron.	Faint trace of iron.	

Bacteriological Analysis.

	A	B
Colonies per c.c. of water on gelatine plates after incubation for 48 hrs. at 22° C.	810	6
Ditto on Agar plates after incubation for 48 hrs. at 37° C.	166	2
B. Coli in 0.5 c.c.	Present.	Absent.
" " 1.0 c.c.	"	"
" " 10.0 c.c.	"	"
" " 500.0 c.c.	"	Present.

Chlorination—Its Origin and what it is.—Filtration, while usually capable of removing 98 per cent. of the total bacteria present in the water, fails to absolutely eliminate them, and if potable water is to be rendered perfectly safe sterilisation must be resorted to. While heat and ultra-violet rays are efficient in this direction they are difficult to handle. Chemical methods—chlorination—are equally effective and comparatively easy to handle. The value of chlorine as a disinfecting and sterilising agent for water supplies and sewage effluents has long been recognised, and the average amount of chlorine required is under one part in one million. With this minute quantity any water may be rendered absolutely sterile without affecting the taste of the water. Moreover, the amount

of chlorine being so small, its cost is insignificant. The mere fact, however, that the quantity is minute was the cause of early failures with the system, and the use of chloride of lime by more or less crude methods has given way to the employment of chlorine gas dealt with in a scientifically accurate measuring instrument.

The use of chlorine gas has many advantages. It is a pure concentrated germicidal agent which does not deteriorate in use, and considerable amount can be stored in a small space. The quantity injected can be determined with precision, the capital cost of the plant is low, and the labour costs negligible.

Sterilisation.—The object of chlorinating domestic water supplies is primarily one of sterilisation. The method would appear to have originated in America at the instance of an officer in the U.S. Army, and in that country this method of water purification is adopted to a considerable extent, often as practically the sole means of purification. The major part of the supply to Greater New York is delivered after storage and chlorination only, the daily supply averaging 500 million gallons. Chicago is supplied from the Great Lakes on a similar basis with water to the extent of 700 million gallons per day. Its use in London is also extensive, Dr. Houston of the Metropolitan Water Board being a firm believer in the system. Some 240 million gallons of water are chlorinated per day in the London area, the bulk of which is taken from the rivers Thames and Lea. The value of chlorination in connection with water supplies to the troops during the war was fully appreciated, and the freedom under those appalling conditions of water-borne disease is ample evidence of its effectiveness as a sterilising agent. Used in various places during epidemics of water-borne typhoid, its value has been proved in the rapid stamping out of the disease, while despite the crude methods previously employed, there is no evidence of any sort that chlorinating water is in the least harmful, and the taste sometimes complained of is either due to overdosing the water—a method both unnecessary and wasteful—or imagination.

Chlorinators.—The commercial form of chlorine used in connection with water supplies is in the form of gas compressed in steel cylinders which contain on an average 70 lb. of chlorine at a pressure of 120 lb. Those cylinders in use at the time are placed on a weighing machine as shown in Fig. 42.

The weight thereof is an indication of chlorine contents of the cylinder at any time. The apparatus shown in Fig. 43 is one type of chlorinator, the "Pulser" type made by the

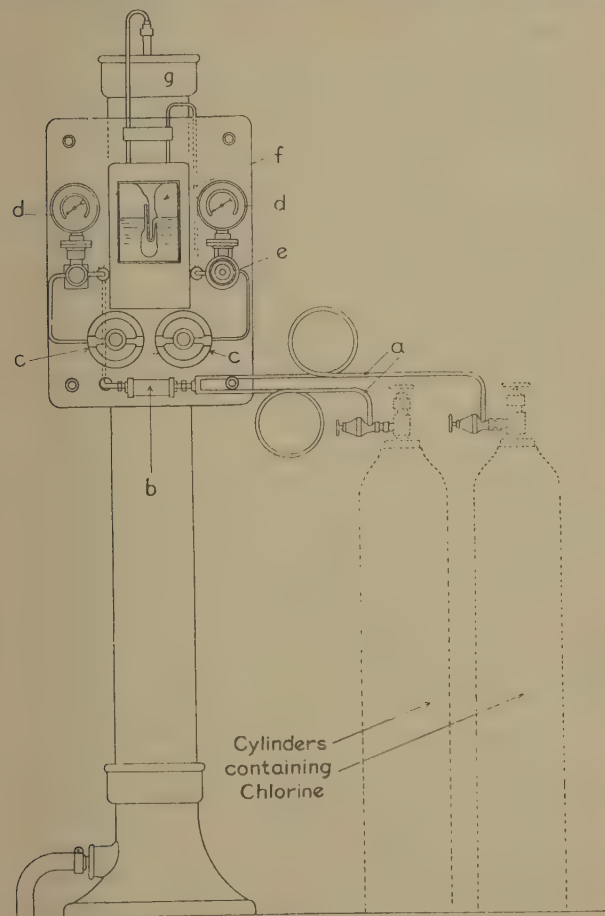


Fig. 42.

Paterson Company. Passing from the cylinder by connecting tubes *a* and a filter *b* its pressure is reduced by valve *c*. With the pressure indicated at *d* the chlorine passes to regulating

valve *e* and to meter *f*. In this instance the meter is of the pulsating type, but there is another type in which the flow of gas is measured by the difference of pressure on either side of a restricted orifice, this arrangement being best suited to dealing with comparatively large volumes of chlorine. To refer to the pulsating meter again, it takes the form as shown of a **U** tube with a connecting branch between the limbs. This **U** tube contains sulphuric acid, which acts as an inert seal between the dry chlorine gas in the instrument and the absorbing water supply. The flow of gas into the inlet limb depresses the column of sulphuric acid until it unseals the small vent pipe so permitting the passage of a definite volume of gas from the inlet to the outlet limb. Equilibrium being established, the column of acid returns till it again seals the vent pipe and the cycle goes on as before. With the regulating valve once set this simple arrangement will pulse for an indefinite time at the same rate. The chlorine so released passes through pipe *g* into the absorption tower, where, being absorbed by the flow of water over broken pumice, it enters the main by pipe at its foot.

Quantity Used and Cost.—The amount of chlorine used is, as before stated, very small, even as low as one-quarter part per million or $2\frac{1}{2}$ lb. per million gallons of water treated, so that with a flow of 1,000,000 gallons per day the hourly flow through the meter would be but $1\frac{2}{3}$ oz.

In other words, 70 lb. of chlorine (the contents of a cylinder) will treat under favourable circumstances 28,000,000 gallons of water. As a general rule one part per million may be taken as a good average dose, the cost of the chlorine at $3\frac{1}{4}d.$ per pound working out at 2s. 6d. When used, therefore, as a safeguard against water-borne disease, its cost becomes practically negligible, while an installation put down with the object of eliminating the labour involved in cleaning pipes, pumps, condensers, etc., would probably pay for itself in a very short time. The saving of coal alone by the better vacuum of a clean condenser might prove considerable. The use of chlorine as a cleansing agent as opposed to a sterilising one is of interest to owners of steam plant who have to make use of dirty water for condensing. The injection of chlorine into such water has been found successful in combating the formation of slimy deposits in condenser tubes, which seriously affect the vacuum and necessitates frequent shut downs for cleaning.

In the case of domestic water supplies there is still a lingering prejudice against the use of chlorine, its addition being regarded as so much dope. Any taste in the treated water is nearly always due to an excess of chlorine—both wasteful and unnecessary. Objectionable taste and odour in chlorinated water is often traceable to an excessive amount of oxidised organic matter, and it may be developed by oxidising agents other than chlorine, the latter merely serving to increase it. That is to say, for any given water there is an oxygen content value above which chlorine may produce objectionable taste, and when this limit is reached some method of oxidation should be used before sedimentation and filtration so as to give an opportunity of the taste to dissipate. Dechlorination can, if necessary, be effected by the use of sulphur dioxide, except, however, when the taste is due to the reaction of chlorine with impurities in the water, and then the addition of permanganate of potash may be necessary.

Water Softening.—The treatment of hard water is generally confined to supplies intended for steam raising or trades purposes. In a number of trades, bleaching, dyeing, paper making, etc., the presence of mineral salts in the water is inadmissible, and modern boiler plants with their high pressures demand the most stringent precautions to exclude any matter which will form scale. At one time softening plants were looked upon as an intolerable nuisance; they were cumbersome, costly to install, and often of doubtful efficiency. Considerable improvement has been made in all directions in modern apparatus.

Water, as is well known, never occurs in its pure state. Rain-water absorbing carbonic acid from the atmosphere possesses a solvent action on carbonate of lime which occurs abundantly in the earth's crust, and the carbonic acid combining with the carbonate forms bi-carbonate. The same action takes place with magnesium. Hence water, and well water especially, will contain bi-carbonates of lime and magnesia together with sulphates, chlorides and nitrates of the same salts and of sodium. Excessive acidity and iron may also be present, but the majority of waters used in industry belong to a class which are alkaline on account of the presence of the impurities mentioned.

Ill-effects of Hardness.—Compounds of lime and magnesia react with ordinary soda and potash soaps, and it is only when they have been precipitated that the soap becomes

effective as a washing agent. A greasy curd is formed which adheres tenaciously to the fabrics, and increases the amount of labour expended in washing. It tends to rot the fabric, and in the case of scouring processes in the textile industries and in laundries involves considerable waste of soap. In dealing with wool, uneven colouring in the dyeing processes and deterioration of the fibre result from the use of hard water.

In leather and tanning works the use of hard water results in a serious waste of tannin and an uneven shade in the finished product. For boiler feeding hard water may bring about a serious state of affairs. The formation of scale gives rise to overheating, while the presence of magnesium chloride and nitrate liberate corrosive acids. In fact, the harmful effects of hard water could be dilated upon at considerable length, but sufficient has been said to indicate the necessity of treating such water, if an alternative supply is not available.

Treatment.—Hardness is divided into two classes—"Temporary" and "Permanent." The first named, being due to compounds retained in solution by combination with carbonic acid, can be precipitated by removing the carbonic acid. This can be done by heating it.

Permanent hardness is not amenable to such treatment, nor is the removal of temporary hardness by boiling usually advisable under practical conditions, and chemical reagents have to be resorted to.

There are several processes, the best known being the lime-soda ash and the caustic soda, and the caustic soda-soda ash methods. The first named is generally the least costly and the more universally applicable, being suitable for the treatment of any water which is alkaline on account of the presence of calcium or magnesium bi-carbonate, and which may contain in solution any of the following impurities:—

- Calcium bi-carbonate.
- Magnesium bi-carbonate.
- Calcium sulphate.
- Calcium chloride.
- Calcium nitrate.
- Magnesium chloride.
- Magnesium nitrate.
- Sodium sulphate.
- Sodium chloride.
- Sodium nitrate.

When the process is carried out in a modern plant little control is required, and the hardness of the water can be reduced to 3° without the addition of any harmful constituent.

The actual cost of the treatment will depend not only upon the total hardness of the water but also upon the relative proportions of temporary and permanent hardness, because the former can be destroyed at less cost than the latter.

The necessary reagents are obtainable at reasonable cost, so that the average cost of treatment need not be more than 2*d.* per 1,000 gallons; under favourable circumstances it may be much less. The use of caustic soda with or without soda-ash is somewhat limited in its application, but for certain waters it may prove convenient and effective. The lime soda-ash process will give satisfactory results with all the impurities tabulated above, and is generally the least expensive. It is a scientific process based on definite and well-known chemical reactions, and this is important in the case of water used for steam raising. No great reliance can be placed upon the so-called boiler compounds; in the first place they prove costly, and there is a tendency to use an excess which may result in corrosion. Properly treated by the method outlined no harmful constituents remain in the treated water; such small quantities of lime and magnesia that do remain in solution form a thin protective scale which prevents corrosion.

Modern Softening Plants—Reagents.—There is nothing new about the lime-soda method of water softening, but the plants previously used were of a crude intermittent type consisting mainly of large settling tanks. They were both costly to operate and inefficient. The modern aspect of water softening involves continuous plants in which the reagents are added to the water in the correct proportion, much in the same way as the alum is added in the case of filtration plants. This mixture of water and reagents is then agitated in a reaction chamber, and after a slow passage through sedimentation tanks the water is filtered, sometimes by rapid filters of the types described or, when this is unnecessary, through wood wool filters.

The actual quantity of reagent is decided after analysis of the water, and simple tests carried out periodically on the treated water serve to control the process. The apportioning gear of these plants is easily adjusted, and provided the reagents used are of definite and uniform composition, little or

no manipulation is required. Most modern plants are designed to make use of "milk of lime" as opposed to the old system of "lime water"; this milk of lime is easily made from ordinary commercial quicklime, although when obtainable at reasonable cost hydrated lime (which is a powder as opposed to commercial quicklime which is sold in lumps) is a superior product, as it is more easily stored, handled, and weighed out, and does not deteriorate so rapidly. Moreover, it is free from unburned limestone which will clog the plant.

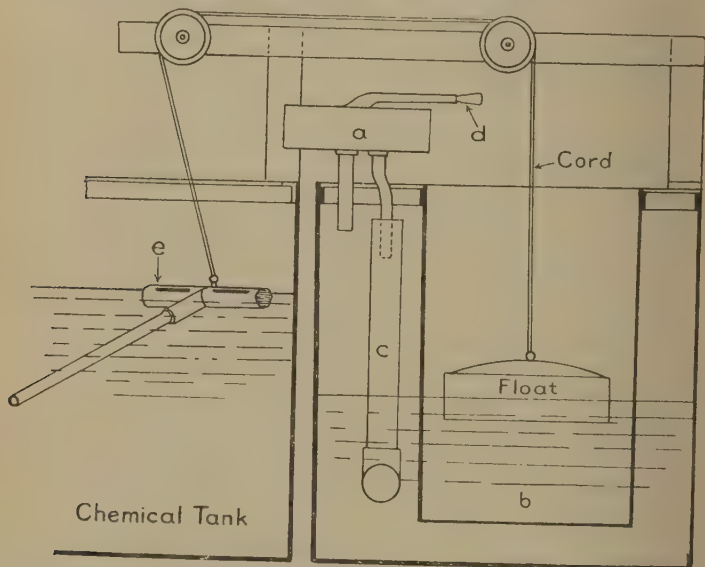


Fig. 43.—Chemical Proportioning Gear for Continuous Softener.

The essential feature of the modern softener of the Paterson, Kennicott, and similar types is the delivery of the water through an automatic proportioning gear which varies the quantity of the reagent added in precise ratio to any variation in the rate of flow, means being also provided for adjusting the amount per gallon according to the state of the raw water. The proportioning gear is usually on the top of the settling tank which forms the main body of the softener. The water so mixed with the reagents flows down a tube in the centre of the precipitation tank or tower and up again to the top of

the latter, the sludge meanwhile being deposited. Proportioning gears operate on well-known hydraulic principle. That adopted on the Kennicott softener is shown in Fig. 43. A small quantity of hard water, from the hard-water box, is passed to a dividing box *a*, and from this box a proportion of the water falls into regulating tank *b* directly below, the rest passing away via pipe *c*. The flow of the water through the two parts of the dividing box is adjusted by means of handle *d*, so that the regulating tank fills up in a definite time, and as the level of the water increases the float gradually lifts. This in turn lowers chemical lift-pipe *e* in the chemical tank, with the result that mixed chemicals are added in a definite proportion to the quantity of water entering the water softener. As soon as the chemicals enter the water the mixture is stirred by the mechanical agitator in the down tank through which the water descends, flowing upwards again outside. The small amount of power required for operating the agitators is provided by a small water motor.

Water Softening by Base Exchange Methods.—This method of water softening, which is exemplified in the Permutit system, possesses several advantages. It resolves itself into filtration through material which possesses the property of chemically absorbing lime and magnesia from the water passing through it.

This base exchange material is a mineral known as zeolite, which is a combination of various metals with alumina and silicic acid. Zeolites exist in their natural state, but it is found advantageous to use artificial zeolite known under the trade names of "Permutit," "Zerolit," and "Doucil," the last named being quite a recent introduction claimed to have certain special properties.

Apart from the fact that water softeners operating by this method are devoid of any moving parts or chemical proportioning gear, they effect "zero" softening, both calcium and magnesium being entirely eliminated. Moreover, this complete removal of hardness is entirely independent of the original degree of hardness in the water. In common with a rapid filter—to which, by the way, this class of apparatus bears a good deal of resemblance—softeners of this type may be operated under pressure or gravity, eliminating in some cases double pumping and permitting of quite small units suited to town and country houses being built. It will be readily appreciated, of course, that after a certain period of

duty the softening material becomes inert, and the great value of the system from a practical and commercial point of view lies in the fact that regeneration can be effected by common salt ; that is to say, a salt solution passed through the filter creates an interchange in the opposite direction. The alternating cycles of softening and regeneration are regular and automatic, the salt being dissolved in an adjacent tank. No sludge is produced and the effluent may be led to the sewers.

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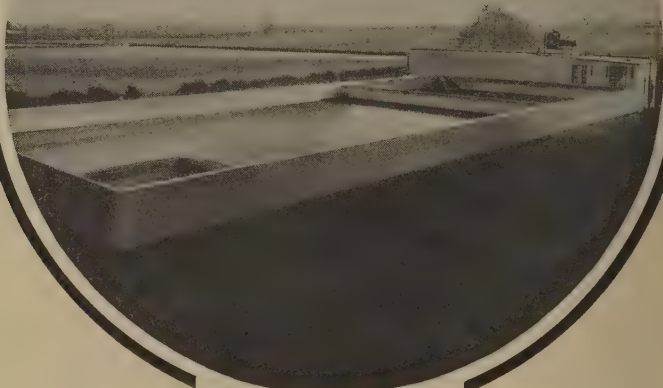
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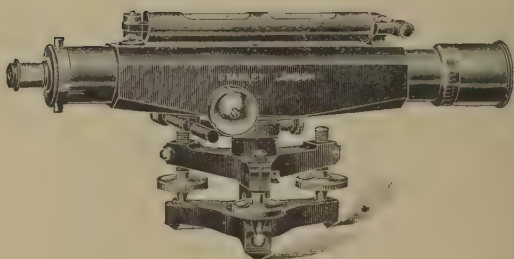
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S0-DRH-723

